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PROTECTION OF BASEMENTS AGAINST FLOODING

TRENDS AND IMPACTS OF DRAINAGE REGULATIONS

A Report Prepared for
Canada Mortgage and Housing Corporation
Under the External Research Program

by

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ABSTRACT

The following research report presents, for the first time, a holistic assessment of the basement flooding problem and all the technologies currently available for its remediation. As part of the Canada Mortgage and Housing Cooperation external research program, Paul Wisner & Associates have reviewed the various courses of basement flooding as well as the state-of-the-art techniques for the identification, prevention and mitigation of the problem.

The main findings and recommendations of the study are as follows:

Findings

1) Because of the lack of national and even provincial guidelines based on state-of-the-art techniques for the protection of basements against floodings (except in Ontario), there are considerable differences between the level of implementation of various solutions, type of models, monitoring standards, etc.
2) Studies conducted in Canada, for the reduction of basement flooding, did not consider the economic benefits resulting from the reduction in treatment plant loading. A cost-benefit analysis is frequently lacking or vague, and the proposed level of servicing varies from one municipality to another.
3) Canadian studies and solutions focus on improvements to the municipal part of the system. Recent USA experience shows that private system connections, with in-house service, may be a major component of the infiltration/inflow contribution.

Recommendations

1) CMHC efforts to improve the use of basements for dwelling purposes should account for the fact that, to achieve the required level of protection, the improvement of the general drainage and sewerage policies would require a very lengthy and complex process. As the only federal agency involved, at present, in all aspects of these studies, CMHC can play a major role in the improvement of drainage policies and the creation of national standards. It is considered, however, that focusing on the private part of the system may be more beneficial in terms of immediate returns. This may include:
    a) improvement procedures for the selection of basement protection devices such as very safe sump pump and backflow valve designs;
    b) review of designs and procedures for the testing of the connection between internal services and the sanitary sewer. This includes the review of plumbing codes, and mainly testing and inspection procedures;
    c) development of a methodology and implementation procedures for the impovement of the private part of the system in areas with substantial infiltration/inflow.

2) There is a need for coordination of efforts for the abatement of infiltration/inflow for relief of basement flooding, as well as for increased treatment costs. Frequently recommended solutions to basement flooding, which are based on improved conveyance in sanitary sewers or storage, do not lead to savings in the cost

of treatment. Federal or provincial funding is not available for flooding relief but can be obtained for pollution abatement in a period of growing national concern over the improvement of the environment. This leads to the following recommendations:

a) Federal or provincial standards for infiltration/inflow studies, including monitoring and modelling, should be developed.

b) Present standards for sanitary sewer design have very small allowances for infiltration and no allowance for inflow. While modern construction techniques are better than those for older systems where high infiltration/inflow values have been monitored, a safety factor against surcharge, which does not exist at present, should be considered to account for aging effects, problems with private connections, faulty maintenance, etc. Since it is obvious that pollution control agencies want to discourage potential infiltration/inflow sources, policing of systems can be assured by monitoring even if pipe sizes are somewhat greater and can probably convey additional flows.

c) Acceptance of very rare overflows from sanitary systems to storm sewers could be considered. Municipalities, which have many overflows from combined sewers and pollution problems caused by frequent storm runoff, could probably accept a sanitary overflow once in 10 or 25 years.

3) Since a significant effort is presently being invested in monitoring and modelling, it would be useful to consider research which may lead to national policies for monitoring and modelling procedures. Several areas are:

   a) hydraulic laboratory and field tests for different types of equipment and national guidelines for monitoring similar to those used in the UK and the State of Massachusetts;

   b) establish a database of the most reliable data obtained, so far, from measurements and use them for the comparison of models, and to develop recommendations for modelling and selection of a modelling package.

4) Since the cost of the various efforts described above is very high and funds are increasingly difficult to obtain, there is a need for a comprehensive analysis of economic aspects related to this problem.

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EXECUTIVE SUMMARY

INTRODUCTION

In the last decade, many Canadian municipalities have been subjected to severe basement flooding. This flooding is characteristic not only of older developments, but also for some more recent ones. Basement flooding and some of its causes were known for more than 100 years. In recent years, however, their occurrence has become more critical than ever because of the increased use of basements for dwelling. The Canada Mortgage and Housing Corporation (CMHC) previously commissioned two studies which addressed flooding caused by surcharge of storm sewers. Findings from the 1984 study (32) identified various engineering alternatives available for alleviating flooding in new developments. These solutions were further analyzed in a second report (1986) (33) in which various types of inlet control devices were tested and their economics realized. Flooding caused by backup of sanitary sewers due to infiltration/inflow were not considered in previous studies. Building techniques were also not analyzed, although the first study reviewed some implementation aspects related to the use of backflow valves and sump pumps which could be considered an auxiliary part of the plumbing system.

CMHC recently commissioned two other studies directly related to the use of basements for dwelling purposes. One deals with building aspects, the other deals with an analysis of infiltration/inflow. The problem of basement flooding was therefore adequately identified by CMHC as requiring a holistic approach in which drainage aspects are interfaced with plumbing and building procedures.

In this report, Paul Wisner & Associates Inc. have reviewed the various causes of basement flooding as well as the state of the art in both diagnosing and applying remedial measures for the problem.

OBJECTIVES

The initial objective of this investigation was to review official drainage criteria and policies. An interim finding of the review was that, while official policies have not been changed for many years, various projects are now implemented taking into account some of the new technologies that have been developed through the last few years.

Accordingly, the objective of this study is to provide a review of the various state-of-the-art techniques for the identification, prevention and mitigation of basement flooding problems caused by storm, sanitary and combined sewers.

METHODOLOGY

Since it was determined that the effective state of the art could not be determined from policy documents which were either outdated or in review, this report is based on recent studies by consultants and discussions with municipal engineers and specialists in various cities. The report presented herein is based mostly on projects conducted in Quebec, Ontario and Alberta, and also draws upon the experience of studies conducted in the USA.

DISCUSSION

a) STORM SEWER SURCHARGE

The status of the implementation of basement protection solutions from separated sewers in Dual-Drainage-Inlet-Control systems studied in the previous two CMCH reports is addressed in Chapter 2. It was found that the use of this solution, which limits the flow to the minor system and prevents storm sewer surcharge, is now more widespread. Many applications, however, are conducted in municipalities, such as Laval, P.Q., where there are no municipal guidelines for their use. In other municipalities, because of lack of criteria, the solution is not applied in a manner which would limit sewer surcharge. As an example, it has been observed that in some situations, while streets have adequate slopes, catchbasins are not always provided with inlet controls.

b) SANITARY SEWER SURCHARGE

A brief historical review of sanitary sewer design, presented in Chapter 3, shows that the need to limit the access of extraneous flows into sanitary sewers has been acknowledged since the beginning of this century. It is also seen that various causes of excessive infiltration/inflow were described in earlier textbooks which also indicated the need for an adequate evaluation of these flows.

Despite warnings in the literature, the practice of the connection of weeping tiles to sanitary sewers continued in many parts of North America and was only gradually discontinued. In some cities in Canada, it is only very recently that the practice was officially discontinued. Most criteria for the design of sanitary sewers have an allowance for infiltration. This is much smaller than observed. The review of measurements shows a considerable variation in the amount of infiltration inflow (I/I).

Chapter 3 describes the different I/I contributions which can be evidenced by these measurements and the general framework for modern infiltration/inflow studies. These studies predict rainfall-related contributions by means of calibrated models. A critical review of models is given in Appendix 1. Systematic I/I measurements with equipment which can provide continuous monitoring information has only become widely used in the last decade.

c) SEWER REHABILITATION

By means of case studies, Chapter 4 reviews various aspects of sewer rehabilitation procedures in Canada. Some studies, for example in the Durham region, did not have continuous measurements for their analysis. Analysis and development of solutions were based on a constant contribution which was varied to reflect basement flooding complaints. These contributions were seen to be several times higher than the design of infiltration rates. With a lack of flow data, the origin of infiltration/inflow and the type of storm (longer or shorter duration) which was deemed to be critical was based on a qualitative analysis of data. In contrast, a very sophisticated routing model, EXTRAN, was applied for surcharge analysis.

In contrast with this approach, other studies described in Chapter 4, such as the ones in Windsor, Edmonton, Metro Toronto, North York, etc. apply measurement data to calibrate an infiltration/inflow model. Models which apply different principless, such as EPA-SWMM, modified versions of OTTHYMO, etc. were used for calibration purposes. While this procedure represents a significant progress in comparison to the qualitative, empirical analysis based on flooding complaints only, the review of these and other similar applications shows that there are several critical aspects.

d) USA AND EUROPEAN EXPERIENCE

A review of several recent I/I studies in the USA and Europe is presented in Chapter 5. Many USA studies were conducted mainly with funds available from the US Environmental Protection Agency (EPA), in connection with relief of treatment plants. (A similar program was conducted in Quebec but, in most cases, without a detailed analysis of rainfall-runoff relations.)

Many European cities have combined sewer systems. As shown by the high infiltration/inflow rates in Chapter 4, many sanitary sewer systems are in reality only partially separated. Some of the European specialists are, therefore, critical about North American efforts to separate sewers in older cities. Nevertheless, many new developments in Europe are built with separated systems. As shown in Chapter 1, the level of protection against flooding for basements is, in general, similar in Europe and North America (1/5-year storms on average). The use of basements for dwelling in invidual European homes is, however, less frequent.

Rehabilitation studies and monitoring in the United Kingdom are conducted based on national guidelines developed by the Water Resources Institute and, in many instances, by specialized contractors. The UK experience for a national coordinated effort as was done for the implementation of new techniques in sewer rehabilitation should be examined. The Scandinavian countries are also very advanced in I/I studies. As an example, a detailed analysis of errors in measurements was conducted in connection with I/I induced by cross flow between storm and sanitary sewers.

FINDINGS

a) Dual-Drainage Design

The problems caused by the more sophisticated nature of the Dual-Drainage Design approach in practical applications are described in Chapter 2 and include the following:

1) the need to improve the interface between the planning, parks and recreational, and engineering departments to ensure an adequate location and design for park storages. This aspect is not so much a technical one as it is an exercise in politics and public relations since various groups (e.g. school boards) have to be convinced that the solution is possible. This is more difficult where the municipality does not have a policy.
2) the need to improve the interface between designers and hydrologists in engineering firms. Some firms involved in urban land development have either not yet obtained the necessary experience in urban hydrology or may conduct, separately, the hydrologic analysis and the design. This type of approach may lead to inconsistencies between assumptions in computations and design details.
3) the verification, by municipalities, of more complex analyses, of the type required by a Dual-Drainage solution, requires specialized staff or assistance from outside consultants. As an example, if several subdivisions with inlet restrictors release their discharge into a trunk sewer, a lack of adequate control of discharge in one of the subdivisions may lead to flooding in another.

The last two aspects mentioned above are related to the gap that still exists between the technical requirements of very advanced design and the experience of engineering staff who are familiar with the more simple and traditional solutions. This has to be considered carefully for any new drainage solutions. Another aspect related to the first point above, namely public awareness, is typical for other solutions related to basement flooding from storm sewers. Major applications in Scarborough, Ont., of park storage relief of surcharged sewers and implementation of sump pumps in Brossard, P.Q., required a significant effort to obtain the cooperation of the public.

b) Aspects of the Canadian Experience

1) Many of the recent Canadian studies include a very detailed analysis of local conditions such as; lot grading (which may not be adequate), quality of pipes, smoke tests, etc. This analysis shows that there are many sources of infiltration/inflow, but that these sources are not easily identified from the results of measurements since monitoring is conducted at only a few locations along the system.

2) None of the reviewed reports has a detailed discussion of errors attributed to the monitoring process. As indicated in Chapter 5 and Appendix 2, there are many methods available for conducting measurements and even the most sophisticated equipment, based on ultrasonic or electromagnetic principles, can give significantly different results.

3) While a calibrated model is used in all these studies for an analysis of remedial measures using a "design storm," there is no uniform policy regarding the return period of this design storm or the level of protection required for basements once remedial measures have been implemented. In some studies, a level of protection was considered based on a relatively simple comparison of damages and costs, and it is proposed that homehowners should provide a major contribution to costs. In other studies, such as the ones in the Regional Municipality of Durham, the verification was done for one of the historic storm events, and it was decided that for another storm (1/100 years) surcharge is unacceptable. In Edmonton, a return period of 50 years was proposed for one area that was experiencing severe basement floodings.

4) The number of storm events used for the interpretation of monitoring is, in general, very small. The infiltration/inflow process is very complex and is governed not only by the precipitation event, but also by antecedent moisture conditions. Consequently, a good understanding of the return period can only be obtained from an analysis of a long series of events. Most studies, however, are conducted with a limited budget for measurements and, as such, may not include data for snowmelt, spring thaw and dry-weather flow conditions, etc.

5) Because of errors in measurements, only relatively large storms should be used for calibration purposes. Some of the studies based their result on smaller storms. The use of a sophisticated

program of measurements in a municipal system is a relatively new endeavour for most municipalities. As a result, the requirements for frequent cleaning and verification of the performance of the measurement equipment are not defined by policies. As pointed out in studies for Metro Toronto and North York, only a few of the measurements could be used in the analsyis.

6) Most of the reviewed Canadian studies recognize the need for two approaches for the elimination of basement flooding from existing and new developments, namely;
   a) measures of the main system such as provisions for local defects and/or storage,
   b) local masurements requiring cooperation with the home-owners, such as; improvement of lot grading, disconnection of weeping tiles and provisions for sump pumps eventually with some assistance from the municipality.

The relative contributions of I/I in the private and public part of the system is not analyzed, although some work on the impact of weeping-tile flows has been done (17). It is generally recognized that the second type of solution is difficult to implement and, for this reason, the emphasis is on the first category which does not require public cooperation.

7) Another aspect, which is not always considered, is the inter-relation between infiltration/inflow studies and pollution control aspects. Measures, for the reduction of infiltration/inflow volume, contribute to the reduction in loading of treatment plants. Therefore, solutions which result in the reduction of

infiltration/inflow, at the source, can be justified, not only on the basis of the improvement of the level of service for basement protection, but also on the basis of savings for treatment. On the other hand, solutions such as storage, where there are overflow structures, can increase the loading of the treatment plant and consequently, as demonstrated in the studies for the Metro Toronto interceptor, significantly increase the required treatment cost. Since most infiltration/inflow studies are conducted for a sewer district only, this aspect cannot be adequately analyzed. A solution which was considered in some studies (Durham) is to provide, for very rare events, overflows in storm sewers. This solution, however, significantly complicates the approval process since, in Ontario, MOE is, in principle, opposed to such discharges, even for very rare storms. (This type of solution was applied in a recent study in the Alta Vista district in Ottawa, in which case approval was sought on a temporary basis.)

8) Study data can be processed as shown for North York in a manner compatible with data management systems such as SIMS.

c) USA Experience with I/I Studies

1) US EPA studies have a mandatory component which is not included in Canadian policies, specifically the verification of infiltration/inflow by a monitoring program after the rehabilitation works have been completed. This post-rehabilitation monitoring program is made mandatory by being included in the conditions of the

federally-funded assistance programs. This monitoring showed that after remedial measures, the reduction in infiltration/inflow volume was, in many cases, less than expected. One of the main reasons is that, contrary to what was expected from general studies, most of the infiltration/inflow is generated in the private part of the system.

2) The descriptions of studies conducted in USA cities in Illinois and California, etc. indicate a major public participation and education effort, legal advice and detailed work conducted on each property based on very stringent verifications by a municipality. This approach was not found in the studies reviewed in Canada.

3) An aspect revealed by the USA experience is that rehabilitation measures taken on some of parts of the system can increase the infiltration/inflow on other parts of the system. This can be avoided only by detailed analysis of all factors, such as groundwater levels, etc.

4) Matrix methods for decision-making such as the ones used in Chicago can be applied for preliminary screening of alternatives.

5) Significant work for the assessment of damages led to relations which consider many factors important in evaluating the various components of basement protection.

CONCLUSIONS

1) Significant progress has recently been made in the area of municipal drainage practice related to basement flooding protection. Many of the techniques are relatively new as this progress is not reflected in most municipal guidelines and policies. Any new technology in urban drainage is applied slowly, cautiously and mainly by means of local initiatives.

2) Because of the lack of national and even provincial (except in Ontario) guidelines based on state-of-the-art techniques for the protection of basements against floodings, there are considerable differences between the level of implementation of various solutions, type of models, monitoring standards, etc.

3) A widely-accepted technique for basement protection against flooding, due to surcharge of storm sewers in new developments, is the Dual-Drainage-Inlet-Control-Device procedure. Although it took approximately 10 years to have this technique applied in other cities where official policies still do not exist, municipal policies for systematic application of this procedure are found mainly in the greater Toronto area.

4) Problems caused by implementation of dual-drainage and other new drainage techniques are the result of non-technical aspects such as public acceptability, difficulties in coordinating various specialities, lack of training programs, etc.

5) I/I-induced sanitary sewer backup resulting in basement flooding problems are complex. Consequently, implementing solutions on a nationwide basis will be difficult. Because of their complexity, a sound approach requires sophisticated measurement programs which may include long-term flow monitoring in addition to smoke or dye tests, TV inspections, etc.

6) There is a need for national standards (similar to the ones used in the UK) or simply guidelines as provided by the state of Massachusetts in the US. This is further reinforced by the review of various studies which showed that some of them did not provide for an adequate duration for measurements, an adequate number of monitoring locations, quality control of equipment, and, none of the reviewed studies conducted an error analysis.

7) Monitoring programs to verify the success of the proposed measures, in studies conducted in the USA, reveal that the amount of reduction of infiltration/inflow is less than expected. Requirements for similar verification programs are usually not part of Canadian practice.

8) Studies conducted in Canada, for the reduction of basement flooding, did not consider the economic benefits resulting from the reduction in treatment plant loading. A cost-benefit analysis is frequently lacking or vague, and the proposed level of servicing varies from one municipality to another.

9) While traditional design of storm sewers was based on the principle of acceptance of failure for a rare storm, legal liability aspects

have not been systematically analyzed. Flooding caused by sanitary sewers should not occur if extraneous flows are eliminated as required by the technical literature for the last 100 years.

10) Canadian studies and solutions focus on improvements to the municipal part of the system. Recent USA experience shows that private system connections, with in-house service, may be a major component of the infiltration/inflow contribution.

1. INTRODUCTION

1.1 BACKGROUND

1.1.1 Drainage Policies

Traditional drainage policies consider mainly the level of service from storm or combined sewers. A review, coordinated in 1973 by the author, indicated that storm sewers in Canada are designed for a return period of two to 10 years, with five years in most municipalities.

If a conduit or minor system is designed for, say, a five-year storm, the implication is that, for a less frequent storm, it will surcharge. Foundation tiles connected to the storm sewer will be under hydraulic pressure and, for a large storm, water will find its way into the basement. The frequency of basement flooding will vary with the frequency of surcharges where the hydraulic grade line is higher than the basement level.

Flooding of a basement is, therefore, a design assumption.

While this is not clearly indicated in drainage policies, any homeowner could incur damages caused by runoff at least at relatively rare intervals. Flooding may also occur because of extraneous flows in sanitary sewers. This occurrence, however, cannot be accepted on a similar basis. Sanitary sewers are designed on the assumption that extraneous flows are very small or completely eliminated.

1.1.2 Basement Usage (Flooding Impacts)

The continuous increase in the cost of housing has led to changes in the use of basements. In new developments, homes have finished basements with family and recreational rooms with carpeting, furniture and washrooms. In such cases, flooding produces increased physical damage at higher restoration costs, as well as greater inconveniences than in the past. These effects have resulted in greater political pressure for remedial measures.

Basement flooding is a major and costly problem in practically all Canadian cities, the northern part of the United States and elsewhere in the world (APWA, 1976). Some Canadian examples are:

In Ottawa, 281 basements were flooded from the storms of June 16 and July 1, 1979. Flooding also occurred in the neighbouring municipalities of Nepean and Gloucester. In the City of North York, 2,151 basements were reported to have flooded during, and subsequent to, the rains of July, August and September of 1986). In the Regional Municipality of Durham, there have been 1,488 flooded basements after the storms of August 27, 1971; August 26, 1986; and March 21, 1980. In Edmonton, severe sewer surcharging and basement flooding occurred during storms of July 1986 and July 1988. Similar flooding problems also occurred in the subdivisions of Lago-Lindo during the 1986, 1987 and 1988 storm events. In the Riverside area of the City of Windsor, and over the years, frequent basement flooding problems have been experienced. The most recent serious event occurred in 1981 when widespread flooding problems were reported. In the Clifton district of the City of Winnipeg, basement floodings were reported from storms of August 1972, May 1974 and July 1977. Similar incidences were encountered in the City of Regina.

1.1.3 Design Deficiencies

The 1984 report for CMHC (Wisner and Hawdur) reviewed many floodings caused by storm sewers, and some of the observations were:

- The reported design frequencies for storm sewers were distributed as follows:

Design return period	$\geq$5 years	>2 years and <5 years	$\leq$2 years
Percentage	33%	48%	19%

- Relatively new sewer systems have been subject to critical flooding. Sixty-five percent of replies indicated a sewer system construction age of about 25 years.
- Flooding often cannot be related to changes in the service area. In 50 percent of older systems and 63 percent of systems less than 25 years old, it was reported that no significant changes in land use or density have occurred. This finding is at some variance with the belief, held by some people, that floodings are, in general, related to land-use changes.
- The frequency of the design rainfall intensity did not correlate with flooding incidence.
- Approximately 64 percent of critical flooding was felt to be related to thunderstorm activity.

In fact, in a large municipality, a storm, or a local "flash-flood," with, say, a 10-year return period may occur each year at another location within that same municipality.

Flooding is not related only to meteorological factors. Several factors contributing to non-precipitation-dependent flooding are:

- Poor hydraulic design of junction and transition manholes;
- Root intrusions in service connections and street laterals;

- Sewers with shallow grades that permit significant local sedimentation;
- Debris deposited by vandals or otherwise entering the collection system.

The complexity of causes of flooding is revealed by the experience of the the City of Scarborough where two major storms occurred in close succession in 1976 and 1977 when over 1,000 homes reported flooding. The City staff identified 23 major areas that have experienced flooding of major proportions at frequencies ranging from once, during a major storm, to almost every year in the previous 10 years or more. The analysis of the flooded areas of houses revealed extensive damage from flooding to furnishings and carpets, leakage through cracks in basement walls and floors, structural damage to basement floors and walls, and settlement of porches, stoops and patios. It was also found that there were sanitary sewer backups into homes and street ponding in excess of 0.6 m.

The causes of the damage in Scarborough were attributed to the following:

1. sewer design assumptions (20 to 30 years earlier) which are no longer valid;
2. few natural open channels or watercourses exist;
3. lack of overland flow routes to suitable outlets;
4. new low points drained into overloaded storm sewers;
5. house roof leaders and footing drains connected to the same private drain;
6. many houses built too low relative to storm sewer;
7. many homes built too low relative to street elevations;
8. some trunk storm sewers very sensitive to flows greater than design storm flows.

Traditional Canadian policies for the level of service of some storm sewers do not provide an adequate protection for basement use, nor do they cover the causes or identify the responsibility areas for all factors in the process.

1.1.4 European Practice

The level of service in Europe is generally not different from that in North America. Most systems in western Europe are combined, although there are isolated pockets of new systems designed with separate sewers. Enquiries into these aspects found examples of service levels determined with semi-empirical uniform rainfall design methods (e.g., Rational Method).

Country	Design Return Frequency
West Germany	. 1 to 5 years
	. Higher values are used for special circumstances such as underpass design.
Switzerland	. Criteria vary with Canton.
	. In general: Small towns - 5 years Medium cities - 10 years Large cities - 10 to 20 years
Spain	. Large cities - 10 years
Finland	. Storm sewers - 2 years
	. Combined sewers - 3 years
	. Higher hazard areas - 5 to 10 years
Italy	. Large cities - 10 years

Table 1-1 Design Sanitary Flow Estimates

Municipality	Residential	Commercial/Institutional/ Industrial
Edmonton	- 0.35 m^3/capita/day - peak factor greater of 2.5 or Harmon's	- no guidelines for commercial and institutional - Industrial, 18m^3/ha/day for trunk sewers (includes peak factor)
Chicago	Avg: 0.389 m^3/capita/day max peak fows-laterals: 1.52 m^3/cap/day mains and trunks: 0.95 m^3/cap/day	not specified
Scarborough	Avg: 0.0013 m^3/sec/ha	Commercial-Industrial 0.0017 m^3/sec/ha Industrial: Not Specified
Ottawa	0.45 m^3/capita/day Peak factor 3 to 6 based on domestic flow	Commercial/Industrial Avg daily flows provided Industrial= Avg sanitary x peak factor + avg. extraneous
Winnnipeg	Avg= 0.272 m^3/capita/day + 22465 l/ha/day Peak (w/o extraneous)= 0.544 m^3/capital/day + 2246.5 l/ha/day Peak (w/extraneous)= 0.272 m^3/capita/day x population x peak factor + peak extraneous for tributary area Peak factor: Harmon or Babitt minimum = 2	Commercial/Industrial: Based on population and residential values or Avg. 16.8 m^3/ha/day Peak hr. 28 m^3/ha/day Industrial: Avg. 33.7 m^3/ha/day Peak hr: 44.9 m^3/ha/day
Calgary	Avg = 0.38 m^3/capita/day Peak Factor = Harmon's	Addressed on an individual basis

Table 1-2 Foundation Drain Connection Practice in Ontario

Municipality	Weeping Tile Connection To Storm	To Sanitary	Remarks	Effective Date
Town of Oakville			provided basement floor above 100 yr flood elev. or 0.6 m above storm obvert; or connect to sump pump or third pipe system	August 1982
City of Burlington			provided no flooding for 25 yr stm or use sump pump	Apr. 77
Town of Markham	√			July 1978
City of Kitchener	certain situations		connect to sump pump, "third pipe" system or outlet not subject to surcharge but not sanitary sewer.	1979
Etobicoke	√			1960
Milton	√			1970
Windsor		√		---
North York		√		1960
Chatham	√		must be connected to sump pump	mid '60s
Ottawa	√			
St. Catharines	√	certain situations	if not possible to connect to storm sewer, connection to sanitary sewers is permitted	

Table 1-3: Examples of Canadian House Drainage Standards for Sump Pumps and Backflow Valves

Municipalities	**Sump Pumps**	**Backflow Valves**
Winnipeg	required for foundation drain (draft) by law	All new homes
Calgary	required in flood prone areas	- Screw caps req'd on sanitary floor drain - all basement plumbing fixtures
Edmonton	- for foundation drain flows allowed to discharge to ground - optional connection to storm sewer via sump pump, if available	Requirement of Plumbing and Drainage Act for services subject to backups
Scarborough	req'd for drainage to storm sewer if basement floor elevation is less than 1 m above sewer obvert	required on storm sewer connections
Ottawa	recommended for new developments	recommended for new developments

It seems, however, that in Europe, litigations for basement flooding are less frequent and expectations for protection are not as high as in North America since the use of basements for dwelling in individual homes is less frequent.

1.2 THE CANADIAN SITUATION

1.2.1 Current Policies

While the contracted intent of this project was to present a critical review based on municipal and provincial standards, a first conclusion derived during the study was that most recent techniques are not described in an explicit manner in current municipal standards. There is an apparent gap between the minimum of 10 years' real-life effective practice and most municipal regulations. Policies are based, therefore, on usage and can vary significantly from one municipality to the other, as shown in tables 1-1 to 1-3.

1.2.2 Relief Alternatives

In the following chapter, we review the status of implementation of various solutions for basement flooding protection.

Two previous reports for CMHC (1984 and 1986) have examined several alternatives for increased protection against flooding caused by storm sewer surcharge. Three main methods were identified:

a) Dual-drainage systems with inlet-control devices (ICDs) -

This method consists of directing overland runoff to depressed areas and using restrictors on catchbasins (ICDs) to limit the access of runoff into the storm sewer.

b) Sump pumps -

Sump pumps are used to evacuate the foundation drain located around the perimeter of the house location. This allows the design of the hydraulic gradient of the surface water to rise to the road surface without any risk of flooding as there is no direct connection between the storm sewer and the dwelling.

c) Storage facilities -

Storage facilities for relief storm sewers may be constructed as either underground tanks for isolated problem locations or as larger, open detention facilities to prevent widespread flooding.

1.2.3 Historical Review and Case Studies

Flooding caused by infiltration inflow will be next examined in the subsequent chapters.

Chapter 3 presents a brief historical review of sanitary sewer design and emphasizes the need to limit the amount of extraneous flows entering the system. Despite repeated warnings in the literature against direct connections of weeping tiles to sanitary sewers, it is only in recent years that this practice has been gradually discontinued in North America. Also, in some Canadian cities, it is only very recently that this practice was officially discontinued. The different I/I contributions, which can be detected by conducting flow measurements on the sewer system as well as the general framework for modern I/I studies, are also discussed. In addition, as an example of such a framework, guidelines for conducting I/I studies, as prepared by the State of Massachusetts, are also described.

Chapter 4 reviews, by means of case studies, various aspects of sewer rehabilitation procedures in Canada. Previous studies, basing their analysis and development of solutions on a qualitative data analysis and flooding complaints, are compared with more recent studies, utilizing more sophisticated modelling approaches applying measurement data for calibrating an infiltration/inflow model. While the latter procedure represents a significant progress in comparison to the qualitative, empirical approach of the past, the review of these and other similar applications shows that there are several critical aspects.

Location	Event	Return Period	Storm Durat.	Predominance of Basement Flooding	No.	Methods of Servicing
Mill Woods	July '78	1:200		A		Double Line, Some Single* Line*
	June '79	1:5		A		
	July 6, '83	1:5		A		
	July 31, '87	1:5		A	82	
	July 5-6, '88	1:5		A		
Laurier Heights	July 17, '86	1:5		A		Double* Line*
Lago Lindo	July 31, '87	1:5		A		Single* Line*
	July 5-6, '88	1:35		A	255	
Dunluce/ Warwick	July 31, '87	1:15		A		Majority- Single* Line*
	July 5-6, '88	1:35		A	121	
North East	July '78	1:5		B		Majority- Double Line, Some Single Line & Combined
	June '79	1:5		B		
	July '81	1:5		B		
	July 16, '85	1:5		A/B		
	July 31, '87	1:15		A/B		
	July 5-6, '88	1:15		A/B	86	

Legend: A. Sanitary Backup
B. Stormwater Seepage

* Foundation Drains to Sanitary System (Prior to Dec. 13, 1988 all)

Table 1-4: Predominant Causes of Flooding in Edmonton

2. IMPLEMENTATION OF SOLUTIONS TO PREVENT BASEMENT FLOODING CAUSED BY STORM SEWER SURCHARGE

2.1 DUAL DRAINAGE WITH INLET CONTROL DEVICES IN NEW DEVELOPMENTS

2.1.1 POLICIES

Basemement flooding in new urban developments, caused by storm sewer surcharge, can be prevented by dual-drainage and inlet-control devices (Figure 2.1a). As the name implies, this concept consists of two drainage systems; the major and minor systems, as illustrated in Figure 2.b.

The minor drainage system comprises roof gutters, rainwater leaders, service connections, swales, street gutters, catchbasins and storm sewers. It accommodates the runoff from the more frequent storms up to the design frequency of the system. The major drainage system, on the other hand, comprises the natural streams, valleys and man-made streets, swales, channels and ponds. It accommodates the excessive flows from rarer events.

In this solution, during rare storm events, storm sewer surcharge is controlled by special Inlet Control Devices (ICD) which limits the flow into the minor system. Overland stormwater flow (major system flow) is stored in a depressed park with a storage volume defined for runoff control protection up to and including 100-year storm events.

While the dual-drainage design concept was introduced for the first time in Denver, Colorado, in 1969, it was only in 1978 that Markham, Ontario, developed, for the first time in Canada, guidelines which introduced the dual-drainage principle as a method to prevent basement flooding caused by storm sewer surcharge.

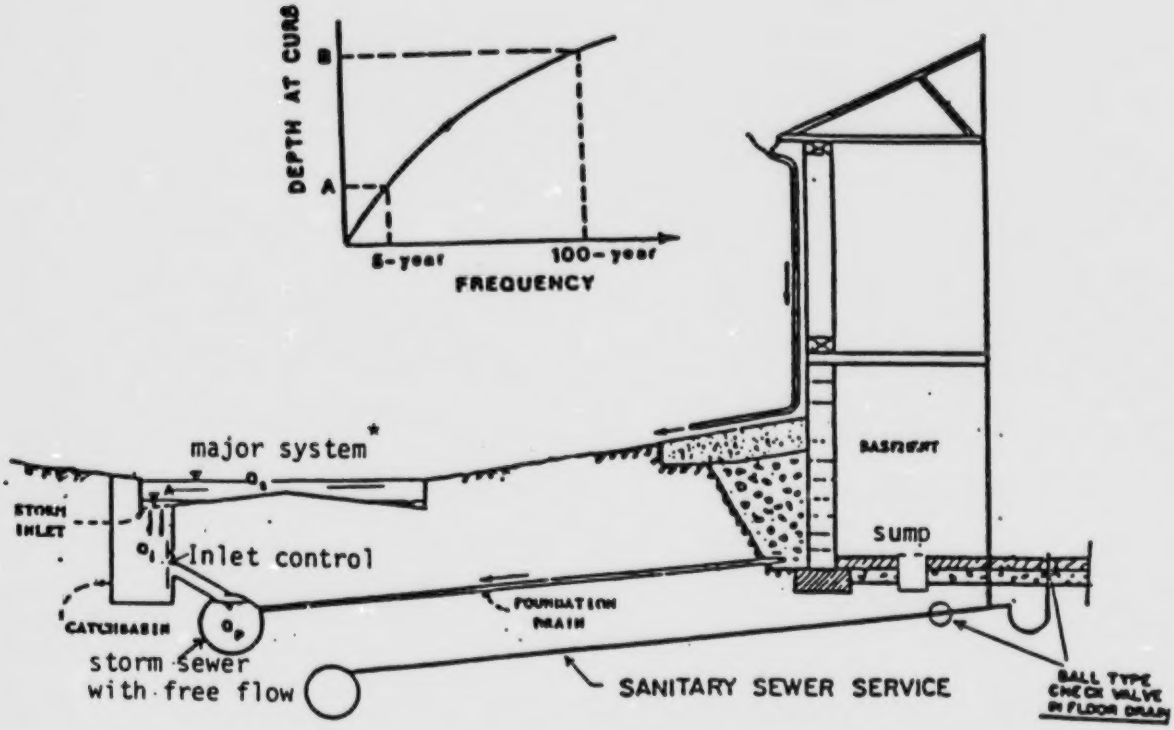
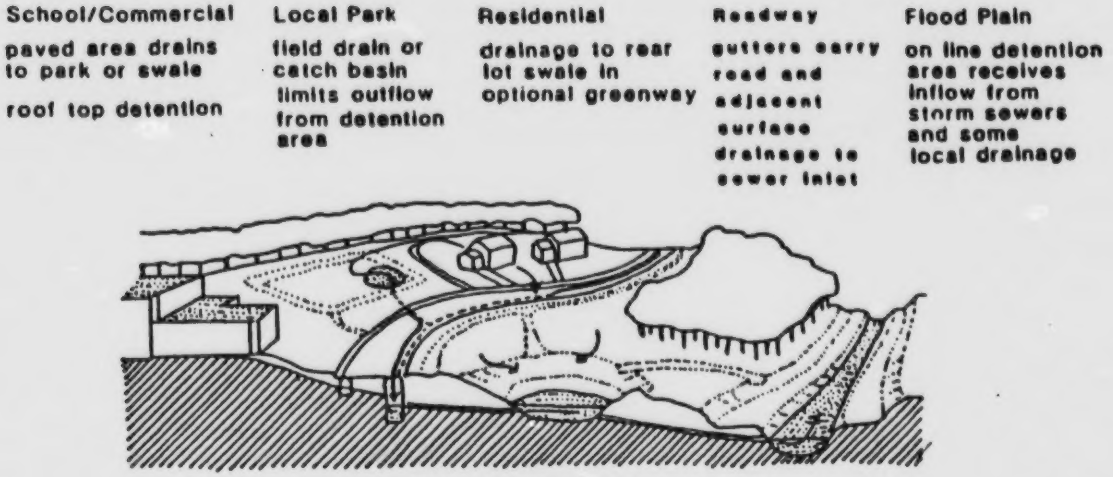
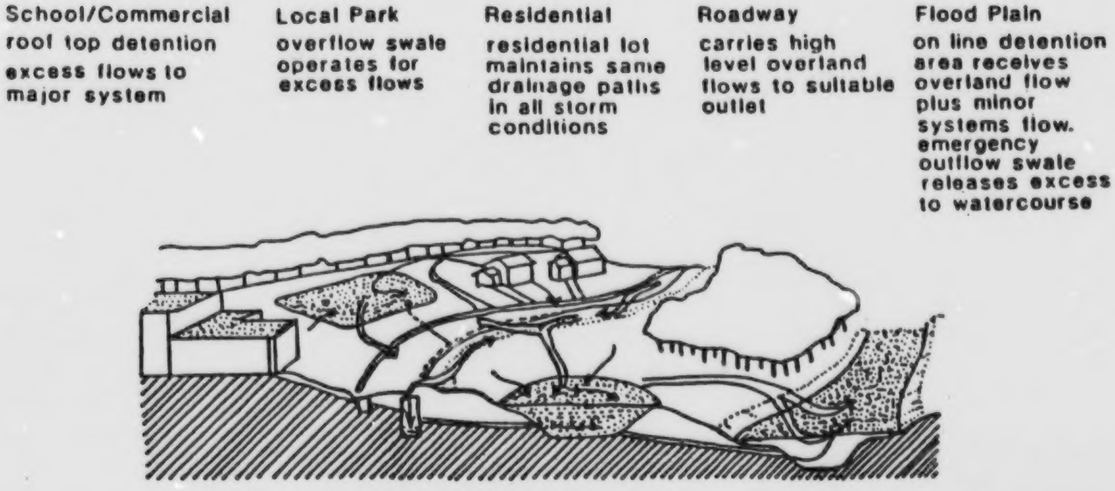

DEPTH AT CURB
B
A
5-year
100-year
FREQUENCY
major system*
Q_s
STORM INLET
Q_i
Inlet control
CATCHBASIN
Q_p
storm sewer with free flow
FOUNDATION DRAIN
SANITARY SEWER SERVICE
BASEMENT
sump
BALL TYPE CHECK VALVE IN FLOOR DRAIN

Figure 2.1a: Protection Against Flooding Caused by Storm Sewer Surcharge by Means of Dual Drainage Inlet Control Devices.


School/Commercial
paved area drains to park or swale
roof top detention
Local Park
field drain or catch basin limits outflow from detention area
Residential
drainage to rear lot swale in optional greenway
Roadway
gutters carry road and adjacent surface drainage to sewer inlet
Flood Plain
on line detention area receives inflow from storm sewers and some local drainage

Minor System Example


School/Commercial
roof top detention
excess flows to major system
Local Park
overflow swale operates for excess flows
Residential
residential lot maintains same drainage paths in all storm conditions
Roadway
carries high level overland flows to suitable outlet
Flood Plain
on line detention area receives overland flow plus minor systems flow. emergency outflow swale releases excess to watercourse

Major System Example

Figure 2.1b: Examples of Minor and Major System Drainage in Dual Drainage Design.

A two-stage approach to drainage in new developments was required:

PHASE 1 - Master Drainage Plans;

PHASE 2 - Detailed Stormwater Management (SWM) Plan;

Dual-drainage concept combined with use of inlet control devices to prevent critical surcharge was mandatory for Phase 2.

These principles were expanded in the drainage guidelines developed by the City of Oakville, Ontario, and the City of Barrie. The last drainage criteria also included numerical examples. Dual drainage with inlet control was also required in policies for Scarborough and Vaughan, in the greater Toronto Area.

The dual-drainage inlet-control concept is also described in the 1987 Urban Drainage Guidelines of Ontario, with main authors Mr. Andrew Brodie and Dr. P. Wisner, which are a synthesis of experience with these previous regulations in the first years of application of the new concepts.

2.1.2 APPLICATIONS WITHOUT SPECIFIC CRITERIA

In Eastern Ontario, the concept was applied for the first time in new developments in Cumberland and in several developments in Nepean in the Ottawa-Carleton region. In the Township of Kingston dual drainage is applied without ICDs since drainage foundations are not connected to the storm sewers. There are, however, very few applications of this concept in Western Ontario (i.e., London, Windsor area), and many municipalities in Ontario did not update their guidelines to include this solution.

Applications of the dual-drainage principle without specific requirements in drainage criteria have been identified in the City of Laval and several other municipalities in Quebec, such as Gatineau and Aylmer. In Western Canada, the first requirement for the dual-drainage concept was introduced in the recent (1989) Servicing Criteria of the City of Edmonton.

Discussions in meetings with municipal engineers in Ontario, Quebec and Alberta show that, even when it is applied, the dual-drainage principle is frequently implemented without requirements for ICDs. In this case, basement flooding caused by accumulation of runoff in low points is avoided, but flooding caused by overloading of gratings of catchbasins is still possible.

2.1.3 DIFFICULTIES IN APPLICATION

Review of many projects and discussions with consultants shows that a difficulty in the application of the principle is when the general slope of the ground may be less than the minimum slope specified by the municipality for a road. The solution to such problems has varied from saw-toothing the road and installing orifice controls on sag catchbasins to installing a grating to capture overland flow and conveying it underground to a safe outlet.

Another problem is that application of the dual-drainage inlet-control principle requires more careful coordination of designs by the municipality than in traditional drainage. An example is a master drainage plan which called for construction of a 2.5 km long trunk storm sewer to service an area of about 300 ha. The sewer was built and an implementation manual was prepared showing where the minor system from each owner's land should be entered into the trunk and the maximum runoff contribution. When the detailed design for the individual subdivisions was done by different consultants, the minor system flows from one parcel was discharged into the trunk one manhole upstream of where it had been planned. The result was that this portion of trunk sewer, and several upstream, could surcharge and cause basement flooding in houses already built. In another situation, the engineers forgot to verify that the orifice controls were inserted in the park drains. If a major storm event had been experienced, the trunk sewer would have surcharged and many basements would have been flooded. Correction of these problems after facility construction is difficult and costly.

The slow implementation of the dual-drainage principle can also be explained by the fact that, since overland flow is discharged in parks, the proper integration into viable urban developments requires a more holistic effort on the part of engineers and planners than has been past practice. This cooperation was not always achieved. For example, Parks and Recreation Departments like to have the park facilities centrally located, but this may not be compatible with the decision to use parkland detention of overland flows.

The application is facilitated in cases such as Glen Abbey Community in Oakville, where the design team took advantage of the Blue-Green concept to create premium lots along the watercourse valleys, channels and detention areas.

As specified in the Ontario Planning Act, parkland represents 5 percent of the developable area. In some municipalities, part of the 5 percent of normal parkland was used for overland flow detention and secondary use of detention. One of the key factors in convincing parks departments to accept the temporary and infrequent detention was that the developers provided completely finished parks as part of the development agreement. It is evident that more technology transfer and a greater public education effort are required to gain a more widespread acceptance for park storage.

The typical detention required for overland flow during a very rare 1:100 year storm is only about 1.5 percent of the land flooded to an average depth of one metre. There are many examples of successful landscaping of parks. In Scarborough, the excavated material was used on site to create toboggan hills and interesting contouring, trees were planted to enhance the park, all walkways were kept above high water level, etc.

In some municipalities, a significant public participation effort was required in order to obtain the approval for the use of parks for detention. In Scarborough, where there are several successful projects, most of the parks considered for detention were located next to schoolyards.

2.1.4 COMPLEXITY OF DUAL DRAINAGE DESIGN

Experience of consultants in areas where the dual drainage is widely applied indicates the following steps:

1. The Master Drainage Plan is first reviewed. Watershed constraints, downstream water level in pipes or receiving stream, watershed areas, land-use assumptions and computer models are checked at this stage.
2. The approved Draft Plan is then be used for providing the design and the street pattern, street slopes and conduit sizes are calculated by traditional methods.
3. The design is next analyzed by the Water Resources Department. Analysis will include the simulation of the street system (Major System) using the OTTSWMM model (Appendix 3) to determine the requirements of ICDs and the Minor System using the OTTSWMM-EXTRAN to determine the hydraulic grade line during five-year, 25-year and 100-year storm events.

4. The limitation of the number of ICDs will be considered to determine the effectiveness of the application. For instance, the use of ICDs should be avoided when overland peak discharge on a street is small (i.e., upstream lateral street). The computed analysis result may require some pipe size changes and other modifications to the design in order to meet these criteria.

Other conclusions were drawn from many projects:

(a) Special care should be focused on the design of the overland route to the designed open space storage facility.

(b) Foundation Drain Collectors are required if the minor system surcharging level is critical to the basement flooding.

(c) Backyard catchbasins should be carefully considered in the computer modelling.

(d) Adjustment may be required in modelling input to consider the roof drains direct or indirect connection to the storm sewer.

(e) Field inspection is very important to ensure that the recommended ICDs are in place to avoid unnecessary basement flooding damage and low-point flooding on streets after the construction period.

(f) The cost of designing according to this principle is greater than for traditional studies. It should be realized that, in addition to a Storm Water Management Report addressing all technical aspects of the design, additional modification to the preliminary design may also be required.

2.1.5 DIFFICULTIES AND TECHNIQUES FOR PROJECT VERIFICATION

As indicated above, verification of these complex projects is a difficult task for municipalities.

Once a MDP is selected, the main runoff control and conveyance facilities are usually built before any land development takes place. As each property owner proceeds with development, many detailed stormwater management studies are conducted by various consultants. Land-use revisions may cause some initial assumptions made at the MDP level to change. The municipal engineering staff is then responsible for checking each Stage II report and verifying its compatibility with the original MDP. This process can be more difficult than conventional drainage design.

For the Town of Markham, a Stormwater Management Data Management System (SWM-DMS) was developed with the following objectives:

a) to retrieve essential information on the drainage system;

b) to update the MDP to include detailed SWM designs as they become available;

c) to enable the municipality to provide design data easily to the various consultants;

d) to verify the operation of the complete system at any stage of the project;

e) to provide an easily accessible reference for future analysis;

f) to provide an independent verification of flows.

The SWM-DMS should provide municipal and consulting engineers with a method of:

1) preventing overload or surcharge of a trunk sewer by specifying a maximum flow from each subdivision;

2) ensuring that the park storage volumes are adequate for detention of overflow (major system) during the 1:100 year storm;

3) monitoring the effects of change to the system, such as land use or imperviousness, catchbasin density, inlet controls and tributary areas;

4) verifying detailed OTTSWMM analysis with a lumped OTTHYMO analysis.

It was also proposed that consultants be required to submit, for each subdivision design, a summary table and a schematic flowchart of the drainage system, summarizing the OTTSWMM analysis. Park storage volumes can also be verified by the system.

2.2 APPLICATION OF ICDs FOR REHABILITATION OF OLD COMBINED OR SEPARATED SEWERS

2.2.1 CANADA

As indicated in the previous CMHC report [33], ICDs can be used as an economic solution for rehabilitation of old storm or combined sewers. The systematic application of this solution in Canada is done mainly in Laval, Quebec [36].

As an example, the traditional approach to rectify the flooding problems of the Montmorency-Brien watershed in Laval, P.Q., would have called for the twinning of 60 percent of the lateral pipes. A solution was developed in order to reduce the twinning of the existing sewer system and also to provide for a higher protection against flooding. This system utilizes inlet control techniques to reduce the flow entering in the existing system. Calibrated orifices are installed in the catchbasins in order to restrict the water flow entering in the combined sewer (Figure

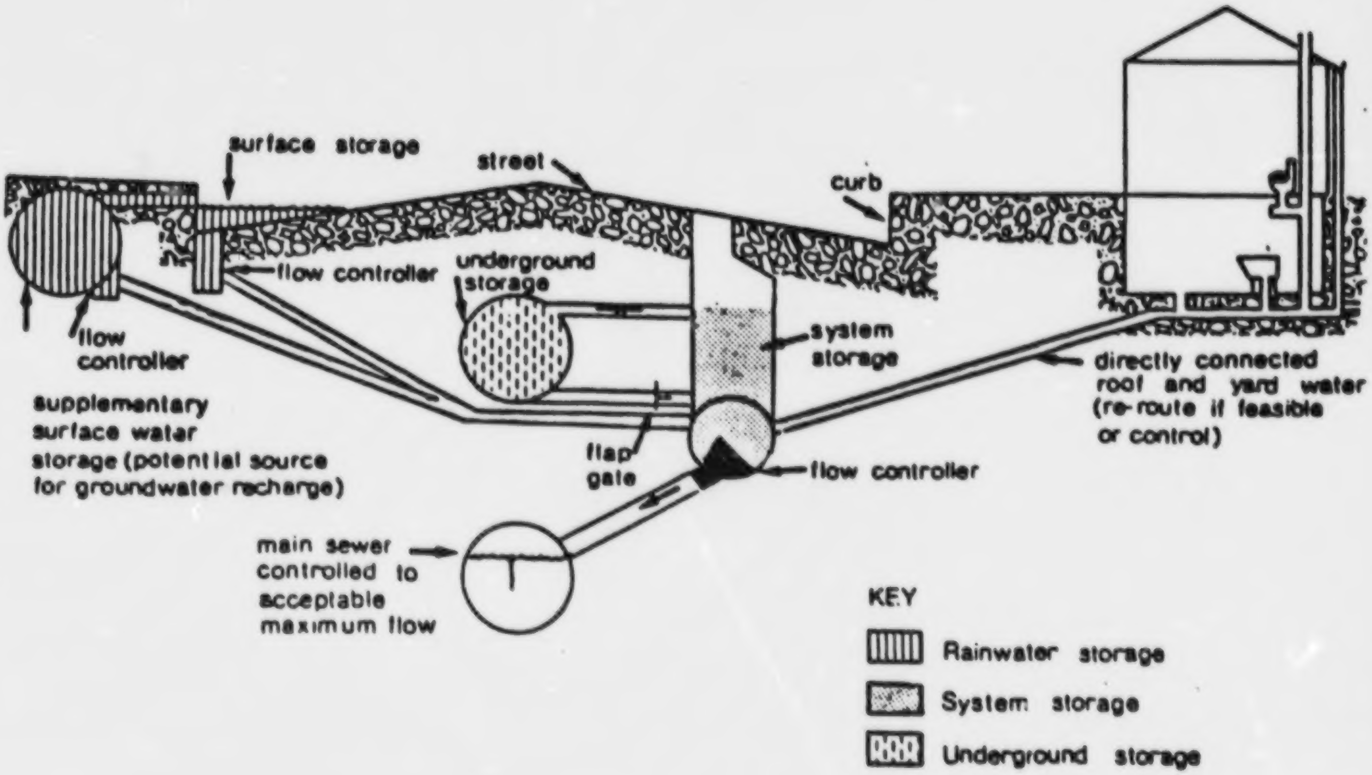

surface storage
street
curb
flow controller
underground storage
flow controller
system storage
directly connected roof and yard water (re-route if feasible or control)
supplementary surface water storage (potential source for groundwater recharge)
flap gate
flow controller
main sewer controlled to acceptable maximum flow
KEY
Rainwater storage
System storage
Underground storage

Figure 2.2: Principle of Application of Inlet Control Devices in Storm Sewer Rehabilitation

2.2). This keeps the hydraulic grade line below basement levels and, consequently, avoids the possibility of basement flooding. This solution uses the streets to carry the water flow exceeding the pipe capacity; hence, the streets are actually acting as open channels in rare storm events, transporting the excess flow which is subsequently captured by new storm sewers. The OTTSWMM model, developed at the University of Ottawa as a dual-drainage and sewer-relief analysis model, was manipulated to generrate and "cut" hydrographs which correspond to existing and proposed systems. In order to compute the surcharge imposed on the systems, the model EXTRAN (Extended Transport) was used in connection with OTTSWMM.

In the proposed innovative technique, the existing system was relieved by means of storm sewers and catchbasin inlet controls. This led to the development of a mostly separated system with considerable advantages:

a) Reduction of the length of proposed new pipes from 10 km projected by traditional design techniques to 2.8 km.

b) The installation of the shallow storm sewer will not interfere with other utility connections and will be less prone to river fluctuation influences.

c) As the proposed system is not directly connected to the residences, flooding by surcharge will not be a problem.

d) Inlet controls keep the hydraulic grade line (of the existing system) below basement levels while still using the existing collector to its full capacity.

e) Implementation of the proposed system will restrict the volume as well as the frequency of combined overflow into the river by about 65 percent.

The traditional approach to this problem was estimated to cost around $7.5 million. This proposed solution with ICDs is approx. $4 million.

2.2.2 APPLICATION OF ICDs FOR SEWER REHABILITATION IN THE USA

Several studies with inlet controls in the Cleveland and Chicago areas were reviewed for this report [24, 31, 37]. Storm water management options for inlet control include the use of:

1. Roof disconnection "with drainage to dry wells/created pervious areas/street catchbasins." Methods to increase pervious/infiltration in urban areas within temperate climates, such as Soakways, Dutch chains and porous pavements are also considered as inlet-control technique.
2. Temporary street ponding, using inlet-control devices (flow controllers) in catchbasins.
3. Flow restrictions in catchbasins and a controlled major system, using speed humps (contoured berms) to direct street flow to less sensitive areas or to areas where surface/subsurface storage is possible.
4. Parking lot storage.
5. Underground storage.

The reviewed study indicated that the inlet-control approach can be immediately applied to provide initial benefits with balance of plan incrementally applied as funds become available. Community investment levels for "inlet control" can be relatively continuous (and always getting a benefit) in contrast to other approaches that are more "lumpy" in nature and in realizeable gains. Communities can expend and realize year-by-year incremental low-level investments and their additive attendent benefits in contrast to a single "lumped" investment taking many years to both accomplish and to realize the benefit.

The benefit of inlet control may be limited due to unfavourable outlet conditions which could reduce full conveyance capacity of trunk sewers. It was found that, in these situations, only a rigorous application of inlet-control philosophy will reveal the true cost-effectiveness of positive relief measures.

It is also indicated that upstream "inlet control" management can have direct positive economic consequence with respect to downstream new treatment facilities. Application of storm water management on "over (storm)/under (sanitary)" systems will reduce the total wet-weather load infiltrating into sanitary sewers from leaky, exfiltrating storm systems.

Application of inlet control on combined sewered systems reduces wet-weather flows, but increase overall "base load" throughout rates as a result of detention storage "bleedback" releases. The optimum mix of storm water management and positive relief implements was determined on a case-by-case basis, considering the total capital and O&M costs of wastewater treatment. It is found that the experience in the USA with strictly sanitary sewer rehabilitation programs has been poor.

Several sewer rehabilitation studies have been conducted in the Cleveland and Chicago areas, where inlet-control techniques have been used. Levels of protection considered in these studies vary between five years and 20 years. While a cost comparison between a conventional (positive) relief sewer system and inlet-control techniques is generally not available, the conclusions indicate that substantial cost savings are realized with inlet controls. As an example, a study for the Ridge Road Area, Parma, Ohio, where frequent basement floodings were experienced, found that for a study area of 9.3 ha (23 ac), a cost saving of $1.35 million was realized by using inlet-control techniques as compared to a conventional relief system. In this study, the city selected a five-year level of protection.

2.3 MULTI-PURPOSE STORAGE IN SCARBOROUGH [39]

As a result of extensive flooding investigation in Scarborough (see Chapter 1), the remedial work was identified and prioritized. One of the highest priority areas included 250 hectares of fully developed land. The main trunk sewer extended for 5 kilometres across this drainage area. Given the fact that an overland flow route could not be established and a new trunk storm sewer would be very expensive, the solution was to create storage areas within the subwatershed.

Some isolated problems were solved by constructing underground stormwater detention facilities within the road allowance. These long cylindrical tanks were designed to store the storm water collected by the road catchbasins and relase it at a controlled rate into the local sewer system. To address the major problem in a separated sewer area, it was determined that by strategically locating and sizing the stormwater detention facilities, the system capacicity could be increased to the point where it could handle a 1 in 25-year storm event compared to the effective 1 in 3-year capacity of the existing system.

Within the top-priority area, five open detention areas were constructed over the seven-year period, 1979 to 1986. Four of the open storage areas are in City parks and one is in the Ontario Hydro corridor. The largest detention facility in a park provides 17,100 cubic metres of storage and involved lowering the park ground elevation by 12 metres.

With this type of approach engineers have succeeded in transforming passive level parks into landscaped, award-winning showplaces.

The design of the park detention areas was carried out in conjunction with the Parks Department and a landscape architect. The parks have sub-drains and base gradients to allow fast drying of the grass, side slopes that can be tractor-mowed and are walkable, energy dissipators at the inlet, remote water-level monitor to alert staff when water is stored, etc.

As all of the detention areas act as a relief to surcharging trunk sewers, there is a control structure on each which diverts flow into the detention area and restricts flow rates out of the detention area during the major storm/runoff events and allows flow rates below a predetermined maximum to bypass the detention area during minor storm/runoff events. The control operates non-electrically due to the possibility of power failure during severe storm/runoff events.

While new developments in Scarborough are now required to provide a level of flood protection for the 1 in 100-year storm event, achieving protection for the one in 25-year event was the best which could be obtained in retrofit cases where no overland flow route exists.

Public groups were understandably concerned about the safety aspects of the open detention facilities. Some of their concerns were frequency of flooding, depth of facilities, depth of water storage, rate of rise of water, drowning of children, side slopes, and rate of draining. Other concerns included loss of existing trees, loss of use during construction (i.e., baseball diamonds), lowering of property values, and whether the bottom of the facility would be wet. The City of Scarborough's Board of Education had similar concerns where detention facilities were to be located in parks that had become so integrated with the schoolyards that, for all practical purposes, they were considered as on site.

The City of Scarborough assembled a design team which included representatives of the Council, the public, the Scarborough Board of Education, design and maintenance staff of the Parks and Recreation Department and Scarborough Works Department Staff, as well as landscape architects and consulting engineers.

Aspects of the open detention facilities found to be important to ensure proper design and operation of the facility were: hydraulic control chamber to reduce the frequency of flooding; good subdrain systems to dry the bottom as quickly as possible; mild side slopes to allow easy egress and lawn cutting. Other recommendations were to reduce inlet velocities to prevent erosion of grass; avoid horizontal bars on grating for safety purposes; elimination of fences around the facility which might restrict egress, etc.

2.4 BASEMENT PROTECTION DEVICES

While many Ontario municipalities do not require mandatory backvalves or sump pumps for new developments, the Quebec practice is different. As an example, in Brossard, a fast-growing suburb of Montreal, the slope of 10 percent created certain difficulties in obtaining sufficient hydraulic head to operate storage facilities for storm water runoff.

Installation, by the developer, of sump pumps in all new houses was required. This would allow the design of the hydraulic gradient for the surface water sewer to rise to the road surface without any risk of flooding as there would be no direct connection between the storm sewer and the dwelling. This measure solved the problem of limiting flows to the system in order to prevent excessive surcharge levels.

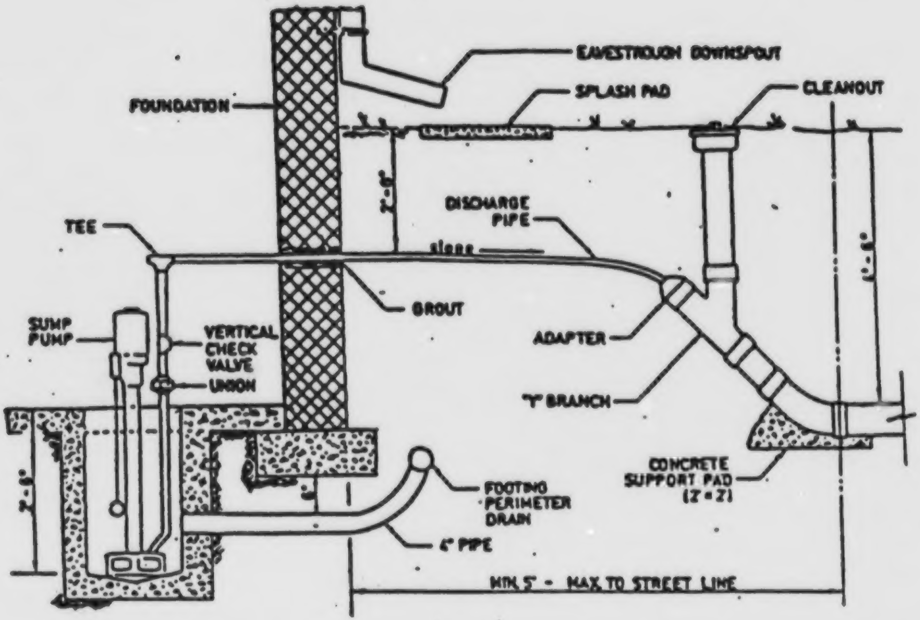

EAVESTROUGH DOWNSPOUT
SPLASH PAD
CLEANOUT
FOUNDATION
2'-6"
DISCHARGE PIPE
slope
TEE
GROUT
1'-6"
ADAPTER
SUMP PUMP
VERTICAL CHECK VALVE
UNION
"Y" BRANCH
CONCRETE SUPPORT PAD (2' x 2')
2'-6"
FOOTING PERIMETER DRAIN
4" PIPE
MIN. 5' - MAX. TO STREET LINE

Figure 2.3: Example of Sump Pump Installation

The key element in Brossard's new approach to surface water management is the installation of individual sump pumps in the basement of each house. Each dwelling is completely independent of the surface water sewer system and, therefore, cannot be flooded by this system even under the worst conditions.

The sump pump is used to evacuate the foundation drain located around the perimeter of the house foundation (Figure 2.3). Three basic conditions must be observed by the developer during building operations:

- A catchpit, with a minimum capacity of one cubic metre, must be installed in the basement in order to house the sump pump. The catchbasin will allow sufficient storage of water from the foundation drain to cover even a prolonged electrical power shortage, and reduces pump cycling.
- All downpipes from roof guttering must discharge, at ground level, at least one metre from the house wall in order to avoid rainwater from the roof being led into the foundation drain.
- When landscaping the lot, the developer must ensure that the ground immediately adjacent to the house slopes away from the house in order that rainwater is shed into the garden and not into the foundation drain.

Approximately 2,000 houses had been built. A random survey of 200 of the owners of properties with sump-pump devices has produced the following information:

Thirty percent reported concern about their pump. "Concern" was defined as: worrying about the possibility of power failure occurring while the owner was away from home, being disturbed by noisy pump operation, believing that the sump pump installation occupied valuable space in the basement and was a hindrance to finishing the basement. (Some of these aspects led to the elimination of sump pumps in Ontario.)

Thirty percent, however, accepted the sump pump installation without question, and twenty percent did not realize that their house was equipped with a sump pump.

Only nine percent reported serious trouble with the sump pump. "Serious trouble" was defined as flooding, continuous operation of the pump, pump breakdown or damage to outside landscaping due to excessive flow from the pump.

A detailed investigation of the 18 cases reporting serious trouble with the installation of their sump pump revealed that, in all but one case, the problem was due to faulty installation by the developer. In some instances, the pump outlet pipe had been connected into the foundation drain; this resulted in continuous recycling of pump effluent. In other cases, the roof drainage had been connected directly to the foundation drain, thus overloading the pump during heavy rain.

In order to prevent this type of situation, an information sheet has been prepared by the Municipality of Brossard explaining the correct installation procedure for sump pumps. A copy of this information is attached to each new building permit and building inspectors pay particular attention to details during their site visits.

Negotiations were also conducted in Brossard with developers regarding the establishment of minimum standards for sump pumps, such as: the installation of high-quality, submersible pumps and the installation of a stand-by pump in case of breakdown. It was also proposed to use a dual-power pump, such as the Home-Guard, which, during a power failure, can operate under standard pressure from a water mains connection.

It is also reminded that in the 1983 study for CMHC, a recent survey of foundation drain connection practice in Ontario, indicated that the majority of municipalities surveyed preferred foundation drain connections made to storm sewers. Both methods of foundation drain flow disposal have benefits and disadvantages that have been hotly debated in many municipalities. Various alternatives for foundation drain connections are compared in Figure 2.4 in terms of flooding control. Wisner and Hawdur in their survey asked municipal engineers about their opinion of the acceptability of backflow prevention - sump pump systems with the following results:

		Opposed	In favour	Not sure
1.	Politicians	28%	40%	32%
2.	Homeowners	56%	18%	26%
3.	Builders/Developers	56%	32%	12%
4.	Municipal Engineers	36%	48%	16%
5.	Consulting Enginers	28%	48%	26%

The success of municipalities, such as Brossard, should be published to encourage the use of these devices.

Some floods-prevention measures, such as backflow preventors, localized storage and sump pumps, are difficult to characterize in terms of a design frequency type of level of service definition. Backflow devices and sump pumps are intended to preclude all flooding episodes. Because they fail with some unspecified and generally indeterminate frequency, they can be used only as additional safety features.

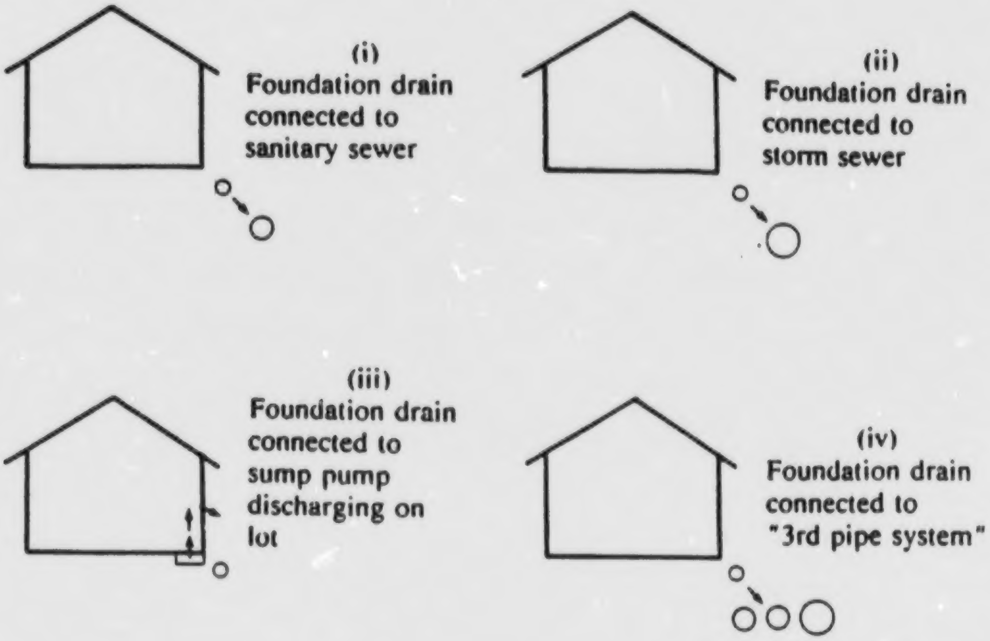

(i)
Foundation drain connected to sanitary sewer
(ii)
Foundation drain connected to storm sewer
(iii)
Foundation drain connected to sump pump discharging on lot
(iv)
Foundation drain connected to "3rd pipe system"

Figure 2.4: Flooding in Various Alternatives for Weeping Tile Connection

	Control
i) Flooding by Sanitary Sewer Backup	difficult
ii) Flooding by Storm Sewer Surcharge	possible
iii) Flooding by Sump Pump Malfunction	limited
iv) Flooding if Outlet of Third Pipe Submerged	possible

Plumbing Arrangements

The City of Regina passed a by-law in 1923 requiring all openings in basements to be kept capped to relieve itself from liability from flooding claims. There were many openings, such as tubs, toilets, etc., which could not be kept capped permanently. It would, therefore, have been necessary to close them manually when the storm occurred. If the house was empty or if the occupants were asleep, flooding could occur. It also puts some strain on a homeowner to have to worry about basement flooding every time a rainstorm occurs. This procedure is, however, used in Germany.

Some municipalities have regulations regarding back valves. Example:

"Any building or structure that is constructed, reconstructed or relocated shall be required to meet the following flood-proofing measures:

(a) An automatic backwater valve or backflow prevented approved by the authority having jurisdiction must be installed in the sanitary and storm sewer lines that service the building or structure where the finished basement floor elevation is at or below the 1:500 design flood level; and,

(b) The finished first floor elevation of the building or structure must be 600 mm or more above the design flood level."

These regulations put the onus of maintenance on the homeowner. Policy, standards for plumbers, inspection of these devices are, however, required.

Similar bylaws have been developed in other cities in Ontario, Quebec, etc.

3. HISTORICAL REVIEW AND CAUSES OF INFILTRATION/INFLOW IN SANITARY SYSTEMS

3.1 HISTORICAL CONSIDERATIONS

A system of drains had been built in the Metropolitan Area of London, England, since the 1500s, but it was developed in a most random manner and had many shortcomings. The cholera epidemic which occurred between 1848 and 1854 resulted in the formation of a Metropolitan Board of Works, which then proceeded to construct an adequate sewer system.

The existing sewers were largely a series of outfall sewers to the river Thames and generally followed along the lines of previous brooks and channels. In addition to causing considerable pollution of the river, it was also noted that, "in times of heavy and long-continued rains, and more particularly when these occurred at the time of high water in the river, the closed sewers were unable to store the increased volume of sewage, which then rose through the house drains and flooded the basements of the houses."

Separate sewer systems in North America were constructed in Memphis, Tennessee and in Pullman, Illinois, in the 1870s. Many of the 4" and 6" diameter pipes became blocked and repairs were difficult because no manholes were provided.

After this experience, the U.S. National Board of Health undertook a comprehensive investigation of both combined and separate systems involving the evaluation of many systems in Europe and the U.S.A. The resulting report, prepared by Dr. R. Hering in 1881, was still described in the early 1900s as "a thorough summing up of good practice." This report indicated that both systems had advantages and disadvantages.

However, in the final analysis, the **preferred choice** was dependant on "the cost of both construction and maintenance, which only a careful estimate, based on the local requirements, can determine."

It can be seen from the literature that the possible causes of basement flooding were well-known even at the beginning of this century. The following excerpts from a book published by Metcalf and Eddy after World War I is of interestas it warns of the possible problems resulting from allowing excessive extraneous flows into separate sewers (underlining by us).

> **Misuse of Separate Sewers** - Annoyance and damage have resulted from the discharge of storm and groundwater into separate sewers. This occurs through the connection with the sewers of roofs, street inlets, and foundation and cellar drains, and, in some cases, even small brooks. Sewers are sometimes so poorly built that they receive large quantities of groundwater that should be cared for by natural channels or storm drains. Even where the sewers are most carefully built to exclude extraneous water, the house connections are often so improperly made that large quantities of groundwater find access to the sewers, thus nullifying the effort and money expended in securing excellence of workmanship in their construction.
>
> The results of such abuses have been serious in many places. In others, however, the ill effects have not yet been so important, because the improper connection of roofs and inlets and the faulty workmanship upon sewers and house connections go on gradually and considerable time may be required for their cumulative effect to become such as to cause damage and demand public attention. The effect of such abuses is destined to increase greatly as time goes on. In one case, a city has been compelled approximately to double the size of its sewage treatment plant, solely because of the admission of water to separate sewers.
>
> The sources of this water are many, including defective building connections, street inlets, perforated manhole lids (particularly within street-car tracks), and abandoned buildings connections which were not sealed when buildings were burned or removed. One of the most important sources in the case cited above appears to have been wet cellars from which water was drained to the separate sewers through the cleanouts in cast-iron soil pipes laid below cellar flows.

> The Lovejoy combination manhole is quite largely used in Boston. The characteristic feature of the design is the storm drain, crossing the manhole above the sewer and provided with a large opening closed with a removable cover, which can be held so firmly in place that there will be no leakage at the joint, even when the drain is surcharged.

It will be noted in the last paragraph that caution is required to ensure that storm water is not allowed to leak into the sanitary sewer even when the storm drain is surcharged as could happen during heavy rainstorms.

A 1953 textbook by Harold E. Babitt deals with the responsibilities of municipalities towards ensuring public health and safety, but also recognizes that sewer design is often based on economic feasiblity.

> **Reliability and liability.** The escape of sanitary sewage into habitable portions of a community is a threat to the public health. Sanitary sewers could, therefore, be so reliable that sewage cannot flow into habitable places. The adequacy of a storm sewer is usually a matter of relative economy between the cost of the sewer and the cost of damages and inconveniences resulting from surcharging the sewer.
>
> Some legal liability attaches to the authorities responsible for the construction or maintenance of inadequate sewers. In general, the legal principle on which liability is based is that the authority is liable which knowingly, either carelessly or deliberately, permits to exist a remediable condition which results in damage to a complainant. If a sewer is constructed and causes damage where no damage occurred before, the construction authority is legally liable. If a remediable, potential cause of damage has been foreseen, or should properly have been foreseen, by a sewer authority, and reasonable precautions have not been taken promptly to avoid damage, the sewer authority is guilty of negligence and is legally liable for resulting damages.

The textbook also warns about the difficulty in the evaluation of infiltration inflow:

> **Infiltration.** Sewers are seldom laid with watertight joints. Since they usually lie below the groundwater level, it is inevitable that a certain amount of ground-

> water will enter unless joints are made, and remain, watertight. Impermeability is an improbability in most sewer construction. In practice, predictions of the amount of groundwater infiltration are not based on formulas, either rational or empirical, but on investigations and on experience.

The 1960 ASCE/WPCF reiterates the need to exclude I/I from sanitary sewers.

> All waters not containing impurities which are actually or potentially objectionable should be excluded as far as possible from the sanitary sewage system. Connection of roof, yard, and foundation drains to the sanitary sewers should be legally prohibited, and such prohibition should be strictly enforced.
>
> The flow rates of sewage for which sewer capacity should be provided must be determined from very careful considerations of the present and probable future quantities of domestic sewage, commercial and industrial wastes, groundwater infiltration, and any other unavoidable contributions.

3.2 SOME CAUSES OF INFILTRATION/INFLOW

Inflow

The principal causes of inflow into sanitary sewers are holes in manhole covers, as well as directly-connected roofs and foundation drains (weeping tiles). Foundation drains are used when it is necessary to reduce the groundwater table around basement walls. This reduces the dampness in basements and prevents structural failure of basement floors caused by hydrostatic pressure from standing water outside the walls. Foundation drains are unnessary when either the structure has no basement, the water table will always be below the foundation footings, or the basement floor is designed and reinforced for hydrostatic pressure. Because of climatic, hydrologic and soil conditions, most new houses in Canada are such that weeping tiles are required.

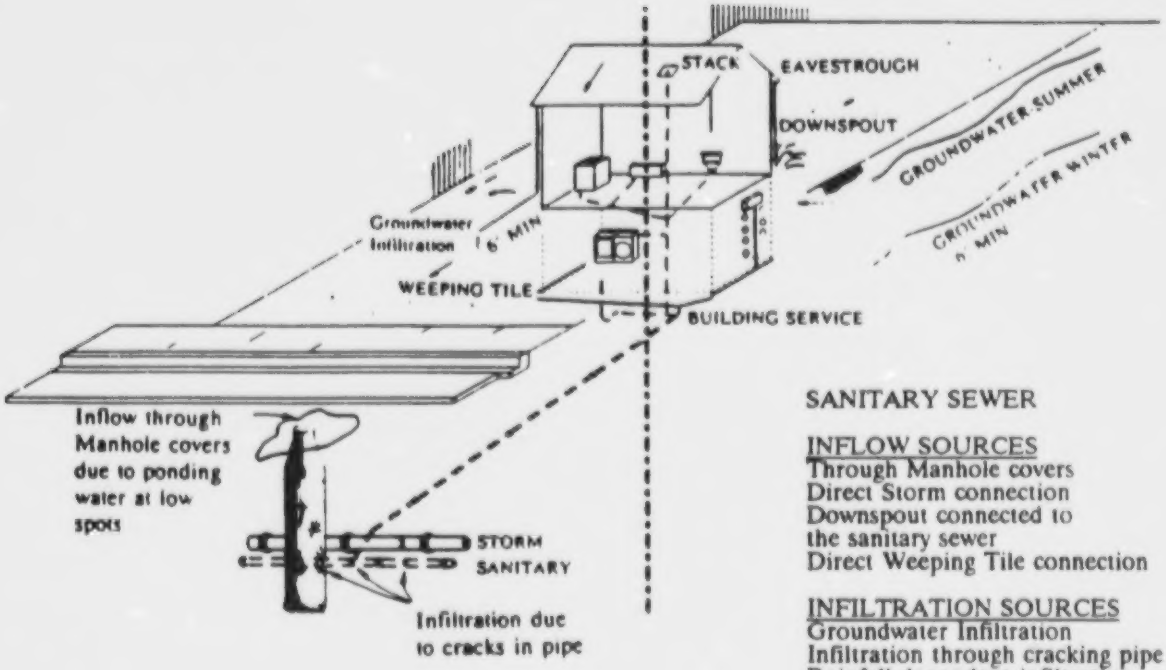
Many variations in house service connections for the disposal of weeping-tile drainage are found in existing developments. Weeping-tile discharge, in recent developments, by gravity to storm sewer, if ever possible, can provide a high degree of property protection when basement floor elevations are such that storm sewer surcharge to the road surface will not cause hydrostatic pressure on building foundations.

Weeping tiles were often connected to sanitary sewers, either illicitly or because a gravity connection to storm sewers is not feasible. This would create an annual and peak of extraneous flow rate that cannot readily be predicted and which are of concern for both pollution and flood control.

Where roof leaders are connected by weeping tiles, the risk of surcharge in an extreme storm is increased. In hilly terrain, surcharge can result in a standing level of water outside the basement with a potential hydrostatic pressure head. When the runoff is conducted to sanitary sewers, the result can be severe overloading of the system and a potential sewer backup into basements. The large-scale connection of roof leaders to the sanitary sewers effectively turns a separate drainage system into a combined or partially combined one. When roof leaders are not directed away from the basement walls using a concrete splash, they can inevitably conduct some, if not all, of the roof runoff towards the weeping drains, depending on the topographic conditions of the house yards.

Infiltration

Infiltration in sanitary sewers occurs when the groundwater table rises to an elevation higher than the sewer invert, and storm water finds its way to the sanitary sewers through defective joints, broken or cracked pipes, improberly-made connections and walls. While in clayey media soil


STACK
EAVESTROUGH
DOWNSPOUT
GROUNDWATER SUMMER
GROUNDWATER WINTER
6' MIN
Groundwater Infiltration
6' MIN
WEEPING TILE
BUILDING SERVICE
Inflow through Manhole covers due to ponding water at low spots
STORM
SANITARY
Infiltration due to cracks in pipe

SANITARY SEWER

INFLOW SOURCES
Through Manhole covers
Direct Storm connection
Downspout connected to
the sanitary sewer
Direct Weeping Tile connection

INFILTRATION SOURCES
Groundwater Infiltration
Infiltration through cracking pipe
Rainfall dependent infiltration

Figure 3.1: Causes of Sanitary Sewer Overloading with I/I (Edmonton MDP)

infiltration might be the minimum freeze - thaw action might leave behind some micro-pores and fissures that accentuate the phenomenon. Another type of rainfall-induced infiltration is called cross-leakage, common where sanitary sewers are constructed at a lower elevation than the storm sewers. In case sewer pipes or joints are not tight, the cross-leakage flow between the two sewer systems will occur. Storm water leaking out of storm sewers will be transported through the backfill and finally intrude into the sanitary sewers.

The various sources of infiltration/inflow as described above are illustrated in Figure 3.1, while an example of the typical components of the total sanitary sewer flow is shown in Figure 3.2.

3.3 TYPES OF INFILTRATION/INFLOW

In more recent studies, several components of I/I have been differentiated:

Groundwater Infiltration (GWI): Groundwater entering the wastewater collection system and building sewers through defective pipes, pipe joints and manhole walls. This component of I/I typically creates a volumetric problem.

Rainfall-Dependent Infiltration/Inflow (RD I/I): That portion of I/I directly influenced by the intensity and duration of a storm event. It consists of storm water inflow (SWI) and rainfall-dependent infiltration (RDI).

Storm Water Inflow (SWI): Water discharged into the collection system from such sources as downspouts, yard and area drains, manhole

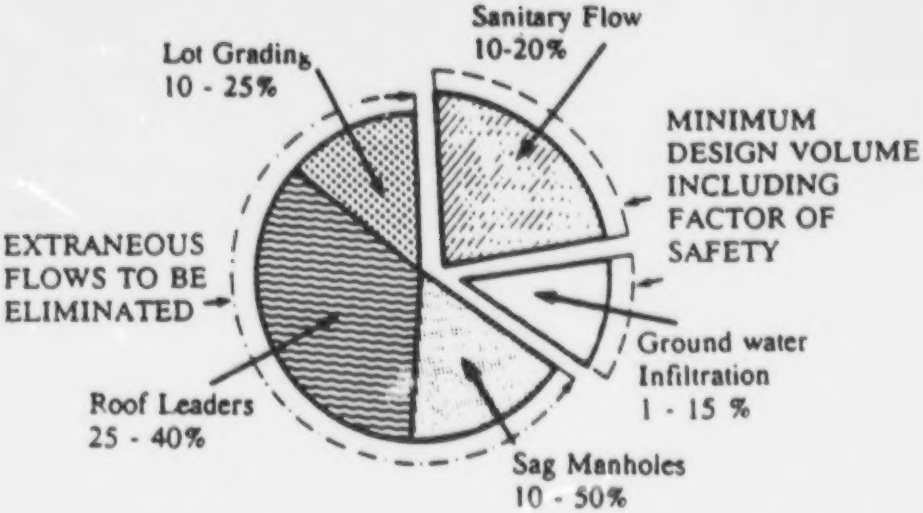


Figure 3.2: Causes of Infiltration/Inflow Identified in the Edmonton MDP

covers, cross connections from storm drains, or catchbasins. This component of I/I creates a peak flow problem. Some sources of SWI are independent of antecedent soil moisture, while others are not.

Rainfall-Dependent Infiltration (RDI): Water that enters the collection system through defects similar to those which contribute to GWI, but in response to rainfall rather than to groundwater levels. This component may be broken into two other components depending on how the water reaches the defects. Rainfall-Responsive Infiltration (RRI) reaches the defects via pipe trenches and foundation drains, and thus may respond quickly enough to affect peak flows. Rainfall-Related Infiltration (RRELI) reaches the sewers only after percolating into the soil, and therefore has a much slower response than RRI. The amount of RDI that reaches a sewer is highly dependent on the antecedent soil moisture conditions, and may be virtually absent when the soil is very dry.

3.4 A GENERAL FRAMEWORK FOR I/I STUDIES

There is a consensus that a successful I/I Study should be performed over a period of several years and will include flow monitoring and cost-effectiveness evaluations at the gross and intensive levels, source detection (smoke testing, dye studies, etc.) and a sewer system evaluation survey (SSES). The basic study methodology can be simplified for the purposes of this review to the following steps:

1. Collect rainfall, flow and sewer data.
2. Describe the relationship between rainfall and I/I.
3. Select design storms and design conditions.
4. Simulate flow in sewers during the design storm.

5. Estimate costs for source detection, rehabilitation and relief.
6. Evaluate the cost-effectiveness of alternative rehabilitation and relief plans.

Flow data at 15-minute intervals are usually collected during several months with long-term and temporary flow monitors. Rainfall data are obtained from recording rain gauges. A review of measurement techniques is provided in Appendix 2.

A cost-effective SSES should first decide whether I/I flows are excessive by comparing cost of transmission treatment of wastes versus elimination from sources.

Sewer data, consisting of pipe diameters, lengths, slopes, elevations, etc. for all the major trunk lines and laterals are obtained from sewer maps and field checks.

Other data included information on land use, sewered areas, and groundwater levels.

For large projets, a data management system was developed to efficiently organize, store and retrieve this large data base for modelling and analysis purposes.

The objective at the next stage in the analysis is to develop a relationship between rainfall and RD I/I for each monitored area. A model of the rainfall to I/I process can provide insight into the sources of the I/I and the effects of soil moisture on these sources. Modelling also allows the engineer to extrapolate from a small observed storm to a large design storm. This is of particular significance since larger events cannot normally be successfully flow monitored because of the surcharging and overflows that often result from such events. Correlation methods are sometimes used, but, more recently, hydrograph models are preferred.

Hydrographs simulated on the basis of observed rainfall data are calibrated by trial and error and a best-fit is achieved (Figure 3.3).

Storm water inflow has a very short time to peak and rapid recession since it is mostly flow from impervious surfaces. Infiltration sources respond much more slowly since they require the flow to travel through soil and enter the sewers through small openings.

In some studies, separate sources of I/I are modelled using separate synthetic hydrographs.

Where SWI is the dominant source, one-component models have achieved good fits to observed flow hydrographs.

The calibration process should be performed for several events at each monitor. The most notable variations in I/I and its distribution into components from event to event can be found to be in response to variations in antecedent soil moisture (Figure 3.4). Storms, preceded by long periods of dry weather, typically show lower overall runoff volumes with more of the flow in components with faster response. This is because infiltration sources are minimized by dry soil conditions (the soil absorbs most of the rainfall), whereas many inflow sources are relatively unaffected by soil moisture.

Limited attempts were made to develop and calibrate a more physically-based model of the rainfall to I/I process that would directly take into account soil infiltration rates and capacities.

A design rainfall event should be selected based on municipal criteria. As further shown, this varies from 5-year to 100-year events.

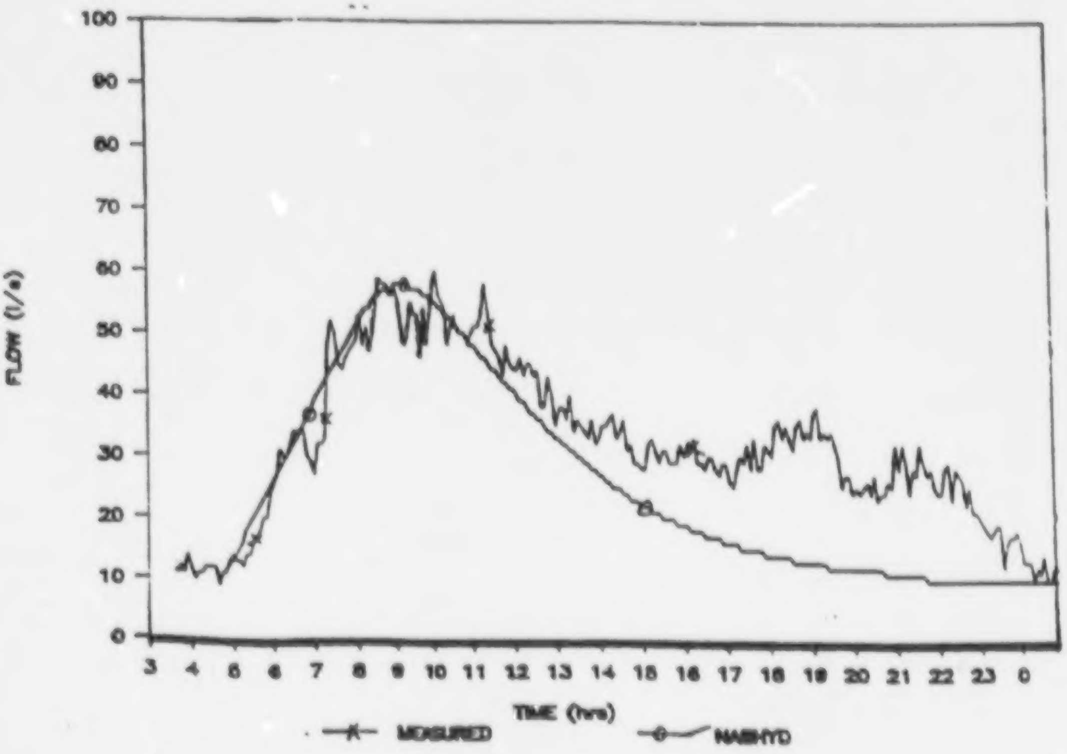


Figure 3.3: Comparison of Measured and Simulated I/I Hydrographs (PWA Study in North York)

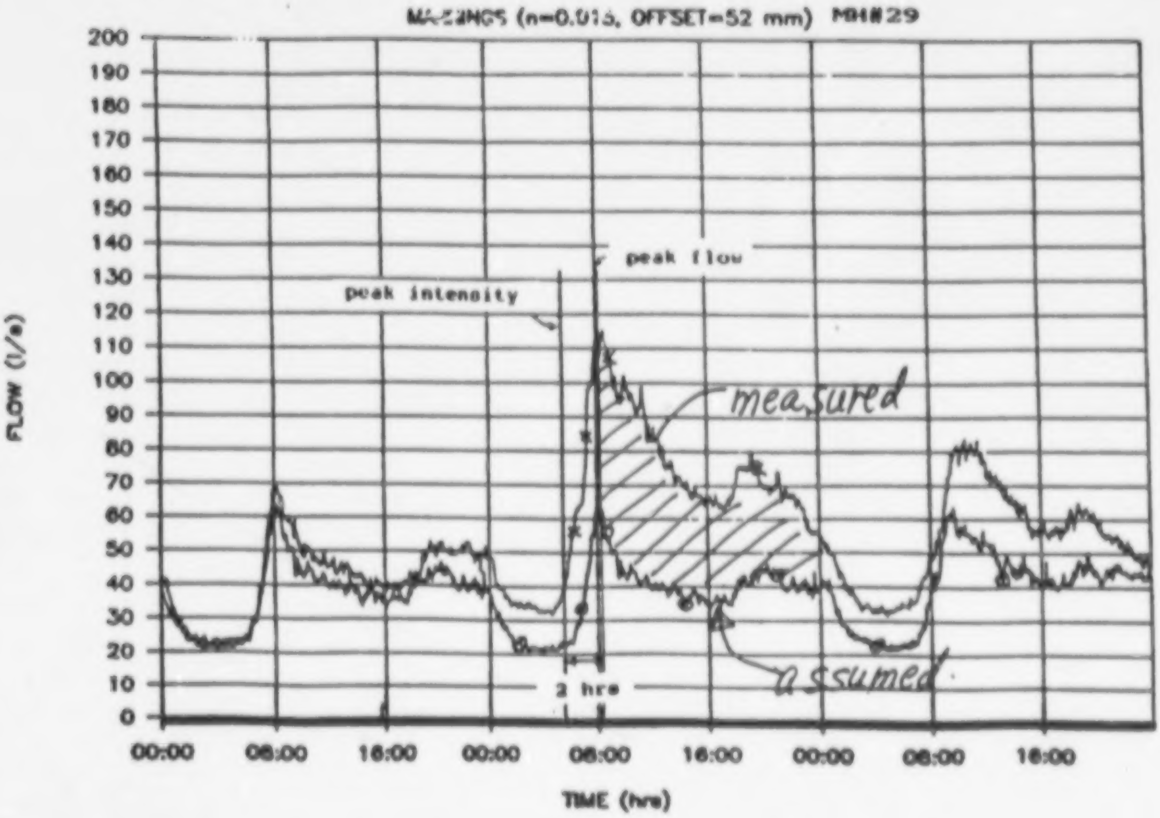


Figure 3.4: Example of Interpretation of Infiltration/Inflow.
Note Approximation Caused by Dry Weather Flow Assumption.

Finally, the design values of GWI and base wastewater flow have to be established. The GWI values can be estimated by subtracting estimated early morning wastewater flow (based on population and land use) from monitored early morning flows.

How to superimpose the design RD I/I hydrograph over the base wastewater flow variations presents a difficult problem. It is conservative to estimate base flow as the average of the peak three hours of a typical high-flow day during the monitoring period. In many cases, the peak I/I dominates the base flow so that the resulting flows were not highly sensitive to this assumption.

The next step is to apply the design storm to the separate areas, generate design flows and route these flows through the community collection system sewers to the interceptor, and, at the same time, size any needed relief sewers.

A Routing Model takes the subbasin flow hydrographs generated by the Base Flow and I/I Models and routes them through the collection system using. The routing method should provide a good compromise between cost and accuracy commensurate with the level of accuracy inherent in the data used to generate the flows. Attention is to be paid to using sufficiently short pipe lengths and time steps so that the spatial lumping of inflows in the pipes does not result in excessive peak-flow dampening or instabilities. The routing model should be verified by simulating an actual event and comparing the flows at several points along the trunk sewer.

3.5 MASSACHUSETTS GUIDELINES FOR I/I AND SSES STUDIES

3.5.1 Purpose and Scope of Guidelines

A very good example of a framework for conducting I/I studies and sewer rehabilitation programs is provided by the Massachusetts Department of Environmental Protection (DEP), "Guidelines for Performing Infiltration Inflow Analysis and Sewer System Evaluation Surveys (SSES). The intent of the guidelines is to provide guidance to communities conducting I/I analyses and sewer evaluation surveys, and to provide assistance such that applicants may meet their grant requirements. While these guidelines are recommended for those communities who intend to perform I/I work on their own, they are mandatory for those communities wishing grant assistance.

The infiltration/inflow analysis is performed to demonstrate the non-existence or possible existence of excessive I/I in the sewer system. If the I/I analysis demonstrates that I/I is excessive, a detailed sewer system evaluation survey (SSES) must be conducted to determine the extent and type of remedial works required. If required, the SSES is conducted in two phases:

Phase 1 - general location of I/I sources,

Phase 2 - detailed location and identification of I/I sources.

3.5.2 Methodology for the I/I Analysis

Inventory of Existing System

An inventory of the existing system is the first major work task and should be performed before conducting any significant field investigations. Information gathered from this part of the study may then be used to develop the monitoring program.

Monitoring Program

The Massachusetts guidelines specify a minimum 10-week I/I monitoring program which includes the following aspects:

Limited Manhole Inspection - At least one or more manholes per system must be inspected in the course of selecting flow-monitoring locations. It is specified that a minimum of 10 percent of the manholes should be entered to verify the general physical condition of the manholes and sewers.

Groundwater Monitoring - The purpose of groundwater monitoring is to provide an indication of the impact of the groundwater level on the infiltration rate into the sewers. In general, the groundwater monitors should be installed every 6 000 m (20,000 ft.), but this varies depending on the specific system layout.

Rainfall Monitoring - Rainfall data are collected specific to the study area during the full monitoring period. Rainfall data enable I/I rates to be determined during different storm events and may be used for conversion to a standard or design storm event.

Flow Monitoring - The guidelines specify that continuous flow monitoring must be conducted for a minimum of of 10 consecutive weeks between March 1 and June 30, starting as soon after March 1 as possible in order to obtain baseflows during periods having high groundwater levels. The monitors should be installed at minimum intervals of one for every 6 000 (20,000 ft.) of sanitary sewer. Again, this number is variable depending on the system layout.

Two-Phase Gauging

The guidelines suggest a single-season, two-phase gauging program whereby the SSES study may be integrated with the I/I monitoring study during the same monitoring season rather than waiting until the next year for conducting the SSES monitoring. Under this approach, the first four weeks of the 10-week gauging program may be used to conduct a preliminary analysis on infiltration and to identify those sewer subareas which demonstrate that an SSES program is warranted.

3.5.3 Sewer System Evaluation Survey (SSES)

The sewer system evaluation survey (SSES) is performed to determine the specific location, estimated flow rate, method and extent of rehabilitation as well as a cost of rehabilitation vs. cost of transportation and treatment for each identified I/I source. The SSES is performed in two phases:

Phase I SSES

Extensive Manhole Inspection - Detailed data on the physical conditon of the sewer system is collected and used for the identification of I/I sources.

Monitoring - Groundwater, rainfall and continuous flow monitoring are conducted in accordance with the methodology described for the I/I analysis (Section 3.5.2). Flow isolation monitoring at selected sewer segments may also be conducted, using instantaneous manual measurements.

Smoke Testing - Smoke testing (Figure 3.5) is used to identify locations of stormwater and groundwater entry into the sanitary sewer systems. These sources may include direct connections

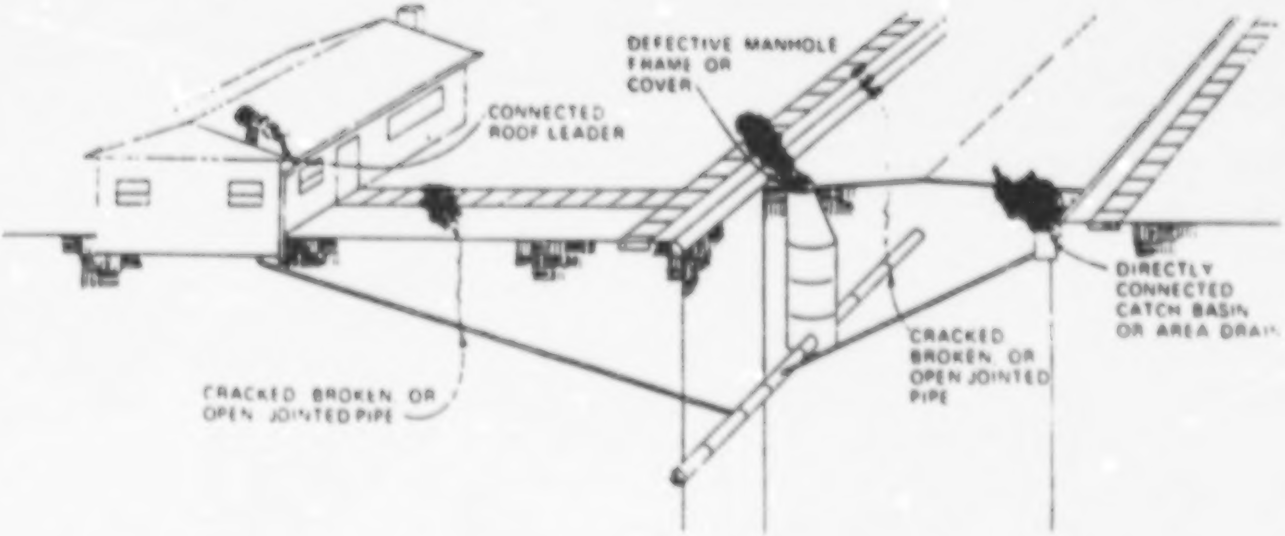

DEFECTIVE MANHOLE FRAME OR COVER
CONNECTED ROOF LEADER
DIRECTLY CONNECTED CATCH BASIN OR AREA DRAI
CRACKED BROKEN OR OPEN JOINTED PIPE
CRACKED BROKEN OR OPEN JOINTED PIPE

Figure 3.5: Smoke Tests to Identify I/I Sources

(i.e., roof leaders, foundation drains, storm sewer inlets, ditches, etc.) and/or indirect connections from storm sewers or ditches which pass through seams in the soil.

The guidelines specify that smoke testing should be conducted during periods of low groundwater (July 1 to November 15) and only when the groundwater level is below the pipe and the ground is not frozen.

Dyed Water Flooding - Dyed water flooding is used to identify sources of inflow to the sanitary sewer. This method can identify both direct connections (foundation drains) as well as indirect cross-connections from storm sewers and ditches.

Internal Building Inspections - House-to-house inspections of private service connections should be performed in areas with excessive inflow. A notification program, using newspaper, radio, telephone and/or door-to-door notices, should be established to inform the public of the overall goals of the inspection. If the property owner will not allow an inspection, he should be dealt with on a legal basis at a later date.

Cost-Effective Analysis - A preliminary cost-effective analysis is used to determine in which areas the I/I is excessive. The guidelines define I/I as excessive if the rehabilitation cost is less than transportation and treatment costs. Areas in which the I/I is found to be excessive are then investigated further in Phase II of the SSES.

Phase II SSES

Groundwater Monitoring - Conducted according to the methodology described in Section 3.5.2, groundwater monitoring is conducted in conjunction with TV inspection for infiltration.

TV Inspection - TV inspection is used to pinpoint the exact location of I/I sources. TV inspection must be conducted only during high groundwater conditions (i.e., between March 1 and June 30).

Dye Water Tracing - Dye water tracing refers to the use of dye water flooding with TV inspection and is used to identify the exact location of indirect connections to the sanitary sewer.

Inflow Balancing - A balance between peak inflow, determined from flow monitoring with peak inflow from inflow source data, must be achieved before proceeding to a final cost-effectiveness analysis. If differences are substantial, the remaining unidentified sources must be identified.

Final Cost-Effectiveness Analysis - A separate cost-effectiveness analysis must be performed for infiltration and inflow. SSES Phase 2 cost-effectiveness analysis for infiltration must consider 300 m (1,000 ft.) sewer segments rather than a manhole-to-manhole approach in order to minimize the potential for infiltration migration. Infiltration migration is defined as the groundwater infiltration to the sanitary sewer which moves from rehabilitated to non-rehabilitated areas. Rehabilitation cost estimates and removal rates for inflow sources can be based on engineering judgment, while 100-percent inflow removal can be assumed for disconnection of direct inflow sources such as roof leaders, catchbasins and foundation drains.

4. ANALYSIS OF I/I-INDUCED FLOODING IN CANADA

4.1 STUDIES IN OSHAWA AND THE DURHAM REGION [25, 26, 52]

Sanitary sewer surcharge and basement flooding experienced in the City of Oshawa are largely due to the inability of some of the sanitary sewers to handle high inflows of extraneous storm water. These high inflow rates result from improper surface and lot drainage, unextended downspouts and foundation drains which are connected to sanitary sewers.

Significant flooding occurred for one of the largest storm events which occurred on June 27, 1971 and produced 5.7 inches (144.78 mm) of rain over a 12-hour period. Unfortunately, the recording rain gauge malfunctioned during this storm and the duration is an estimate from standard gauge measurements. The return frequency for the June 27, 1971 storm, based on the above information, is in the order of 1 in 100 years. Remark: Although this event resulted in widespread basement flooding in the study area, it was not used for model verification, since the time distribution of the rainfall is not available and the severity of the rainfall was significantly higher than the design level for both storm and sanitary drainage system.

The second outstanding storm event occurred on September 4, 1981. Again, the recording rain gauge failed and the duration and intensity of this event is unknown. This storm is estimated to have a 25- to 50-year return period based on the 54.6 mm (2.15 inches) obtained from a standard gauge. This event was used for model "calibration."

However, most flooding from such an event would result from the poor overland flow drainage rather than inadequacies in the sanitary sewer system after implementing the recommended improvements.

Flow measurements were not available. Studies of surcharge, conducted in Oshawa in 1982 by MacLaren, considered initially an infiltration rate of 4,000 gal/ac/day (44,932 l/ha/day). It was assumed that this value reflects flows entering pipe joints, cracks, etc., and that foundation drains are connected to sanitary sewers. This was considered a high design infiltration rate compared to that used in other localities.

The primary task of the study was to use a computer model to analyze the hydraulics of the sanitary sewer system in the study area for both existing and ultimate conditions.

Sanitary flows were estimated according to the Region's current design criteria. Modelled stormwater inflows were determined by attempts to reproduce with the model the observed basement flooding and recorded water levels in the system for the storm of September 4, 1981. The calibrated storm water flows were added to the ultimate design flows for the system to determine the relief requirements. It was found that storm water flows, in some areas of the system, were seven times the design infiltration rate of 4,000 gal/acre/day. It was determined that these large inflows were the result of direct connections of roof leaders, poor lot grading which permits surface drainage (and roof water discharges) access to the foundation drains and poor road drainage which causes the inundation of manhole covers in depressed areas.

The level of surcharge and extent of historical basement flooding were generally well reproduced by the computer model, assuming increased extraneous flows in localized areas. The estimated rates of additional extraneous inflow (i.e., in addition to the 4,000 gal/ac/day design rate) resulting from the rainfall of September 4, 1981, vary from location to location, but range from three to six times the design rate of infiltration (i.e., 12,000 to 24,000 gal/acre/day, 134,800 to 269,800 l/ha/day).

The relief measures chosen for each area were modelled in detail to establish the most effective relief alternatives.

Areas receiving highest priority with respect to relief should be based on the location with highest number of reported basement floodings and with the highest frequency of occurrence.

To test the sensitivity of the relief system, the occurrence of larger extraneous flows than those which occurred on September 4, 1982, an increase of 25 percent to the modelled extraneous flows was analyzed under ultimate conditions.

The improved system may still be susceptible to flooding from very severe storms such as the one experienced on June 27, 1971. This storm was estimated to be in the order of a 1:100-year event. Events of this magnitude produce an amount of runoff which greatly exceeds the practical design capacity of sanitary sewers. The storm drainage system (sewers plus overland flow routes) must be designed to convey these flows if basement flooding is to be precluded during these very large storms. Unfortunately, in the study area, the existing storm drainage system capacity is constrained by inefficient road grades that prevent the safe outlet of major storm flows. This condition, coupled with poor lot grading and roof leader connections to the sanitary sewer system, make it impossible to guarantee that basement flooding will not recur in the future during very large storm events.

Several years after completion of the previously described study, on the evening of August 26th, 1986, a new severe rainstorm occurred within the Regional Municipality of Durham, preceded by heavy rains the previous week. As a result, 861 basement flooding reports were registered within the Region of Durham in Pickering, Ajax, Whitby, Oshawa and Newcastle.

The study by Simcoe Engineering developed a system whereby each flooding report could be categorized as to whether it was related to sanitary sewer flooding or to some other cause. To this end, all flooding reports were categorized as being either 'dispersed' or 'localized.' 'Dispersed' reports are defined as those which are not related to flooding of the sanitary sewer system. Localized reports are defined as those which are related to insufficient capacity in the sanitary sewerage system to handle the infiltration flow generated during the storm of August 26/27, 1986.

Questionnaires were distributed and answers catalogued according to a simple identification code. This identification code utilized abbreviations of the municiaplity, drainage area and address at which basement flooding occurred. In addition to the identification code, the remaining fields within the inventory file were utilized in such a way as to identify:

i) type of basement flooding report ('dispersed' or 'localized');
ii) extent and general description of the flooding; and,
iii) possible factors which contributed to the basement flooding.

The study suggested that, for future storm sewer construction in areas where only open ditches exist now, individual on-site work must be undertaken to separate the foundation drains from the sanitary sewer connection. It is estimated that the cost per household for the construction of an additional service connection and changes to necessary internal and external plumbing could range between $1500 and $2000.

The study considered that, in areas where sewer separation has already occurred, and foundation drains still remain connected to the sanitary sewer, the only method to achieve complete separation is to construct an additional third pipe for the foundation drains. For typical subdivisions, the cost for a third pipe system could be as high as $5500 per household. Of this cost, approximately $3500 would relate to municipal property (including the third pipe and service connections), and an additional $1500 - $2000 would be attributed to private property service connection and changes to the plumbing within the household.

This study utilized the PCSWMM3.2c EXTRAN module (a dynamic version of the EPA SWMM model) as an analytical tool to identify the areas within the sanitary system in which surcharging was occurring to the extent of causing basement flooding. The EXTRAN module was used because of its unique capability to calculate the surcharge levels in the manholes.

Since, again, there were no flow measurements, the only data which were available for model calibration consisted of observed water level elevations in flooded basements, and any associated sewage pumping station peak-flow data. The Region collected limited field data on peak water levels in certain manholes. Consultants felt that the means by which these data were collected were questionable, and the overall results were not always consistent with the water levels observed in the basements. Therefore, this data source was only used for general comparative purposes.

At certain areas within the system, the simulations could only reproduce the observed hydraulic grade line using unrealistically high infiltration rates. These rates could not be supported by any previous studies or any other reliable means. In cases such as these, it could only be assumed that a factor which could not be accounted for, such as a blockage within the sewer, had occurred.

Since rainstorms which affect surcharging in the sanitary sewer system, as a result of foundation drains, are significantly different than most storm events which cause high levels of flow in storm sewers, it was considered to utilize existing storm sewers for very rare overflow purposes in order to provide protection to basements against sanitary sewer surcharging. This, however, is not acceptable to MOE.

If, for some reason, it is necessary that foundation drains be connected to the sanitary sewer system, it is recommended that the sanitary sewer systems be designed for a capacity of no less than 225 m^3/ha/d (20,000 gal/ac/d) for wet-weather infiltration. This recommendation is based on results of the "calibration" process which was carried out as part of the Report.

Some of the remedial measures which were proposed included:

a) Diversion of local sanitary flows to an adjacent trunk sanitary sewer having excess capacity, at a location upstream of an existing connection.

b) Increasing pumping station capacities at a downstream pumping station, after verifying that the increased flow downstream would not result in additional deficiencies.

c) Emergency overflow pumping facilities to be used on an interim basis in the event of occurrence of high flows within the system.

d) Temporary overflows to adjacent storm sewer facilities, to be utilized at an infrequent level of recurrence.

e) Relief sanitary sewers to increase the hydraulic capacity of the existing system, again verifying that the increased flows downstream can be adequately handled.

f) Reconstructing and upgrading existing trunk sanitary sewers to increase the hydraulic capacity and to divert flows from areas which are presently experiencing capacity problems.

g) Disconnection of foundation drains from the sanitary sewer system and reconnection to the storm sewer system in areas where storm sewer installation is being planned.

4.2 STUDIES IN WINDSOR [41]

Over the years, frequent basement flooding problems have been experienced in the Riverside area due to surcharging of the sanitary sewers. The most recent serious event occurred in 1981 when widespread flooding problems were reported. Additional prolems include excessive wet-weather flows reaching the treatment plant, which results in numerous by-passes to the Little River, and the occurrence of overflows to the Detroit River due to overloaded sanitary sewers. The main cause of these problems is the entry of excessive flows to the sanitary sewers via improper direct connections, foundation drain systems and severe infiltration conditions.

Most of the sewer system was constructed over 40 years ago. Typical of older systems, open joints and cracks have developed along the system which permit inflows and contribute to the surcharge problems. Badly deteriorated storm and sanitary service connections allow runoff to exfiltrate from the former into the latter during wet-weather conditions. Typically, the storm service connection lies above the sanitary service connection and receives roof runoff via the rainwater downspouts; the foundation drains are generally connected directly to the sanitary sewer.

Prior to the hydraulic analysis, a structural performance phase incorporated several factors, such as structural integrity, soil types adjacent to the pipe and the frequency of surcharging experienced.

Five condition grades were defined, as follows:

Internal Condition Grade	Structural Condition
5	Collapse or collapse imminent
4	Collapse likely in foreseeable future
3	Collapse unlikely in near future, but further deterioration likely
2	Minimal collapse risk in short term, but potential for further deterioration
1	Acceptable structural condition

Soils were divided into three types according to the risk of groundwater being lost into the sewer; these are as follows:

High risk . silts, silty fine sands or fine sands
. medium to course sands

Medium risk	. low plasticity clays . fine to medium gravels . well-graded sandy gravels
Low risk	. medium to high plasticity clays . all clays if sewer was constructed by tunnelling
Daily	. usually tidal
Frequently	. to a minimum of 1.5 ft. above the soffit at least once a year
Rarely	. to a depth of less than 1.5 ft. at intervals greater than once a year.

The combined effects of surcharge and soil type were next used to modify the initial internal condition grade.

Generally, only the sewers with a structural performance grade of 4 or 5 require immediate attention, and sewers with a grade of 3 should be inspected regularly, and remedial action taken if further deterioration occurs.

An analysis, applied to the 77 pipe segments, identified as critical sewers in the Riverside Phase I area, indicated that only 13 were assigned a structural performance grade of 4, and of these, five require only local repairs. The remainder were in the 3 or less category.

Another part of the study was a smoke and dye testing, conducted by City of Windsor staff.

Dye was introduced at the eavesthrough of the individual buildings and the downstream manhole of both the sanitary and storm sewer system was monitored for evidence of the dye. Smoke testing was carried out along the sewers to identify any improperly connected catchbasins.

The results of the investigations revealed that, within the separated area, 89 houses, or 11.6 percent of the total, have rainwater downspouts connected to the sanitary sewer system. Also, a total of 50 catchbasins were directly connected to the sanitary system. Effectively, some 36 acres, or 17 percent of the separated area, contribute runoff directly to the domestic sewers.

For rainfall and flow monitoring, two rain gauges were installed to ensure that spatial variation in the rainfall would be recorded. For flow measurements, four of the Montedora Whitney SystemQ instruments were installed permanently at designated locations, and the other two were moved to several different locations, as deemed necessary. A description of the SystemQ is provided in Appendix 2.

With the SystemQ logger in place and operating, a velocity profile was taken, using a portable velocity meter. The investigations revealed the need for velocity calibration. A significant variation in the flow coefficient value of the System Q, from a low of 0.65 to a high of 1.50, was observed.

The flow conditions recorded during the five rainfall events confirm the fact that surcharging occurs frequently in the sanitary sewer system, and illustrates the severity of the problem. All the storm events produced, however, rainfall depths less than those associated for the two-year return period.

The computer modelling was carried out using the OTTSWMM program for the runoff simulation and the EXTRAN program to analyze the sewer hydraulics. Of the five recorded storms, three of the events were used for calibration and two for verification of the model.

Isolating the individual contribution from sources other than the improper direct connections (i.e., infiltration, foundation drains) was not investigated. Rather, the combined contribution was considered as a lumped quantity. Measurements indicated that approximately three percent of the total rainfall depth entered the sanitary sewers.

A comparison of measured and simulated hydrographs indicated that the computer model adequately reproduced the hydrologic/hydraylic response of the basin.

The performance of the existing system was next assessed by simulating the rainfall-runoff process for three design events, the 2-, 5- and 10-year storms. The hyetographs (i.e., rainfall distributions in time) for these events were derived from rainfall data measured by Environment Canada at the Windsor Airport gauge.

As expected, the results indicated that the sanitary sewer system is presently heavily overloaded, resulting in excessive surcharging.

On one of the trunk areas (Wyandotte), there are some 89 houses with rainwater downspouts connected to the sanitary sewers, and 50 catchbasins which are also directly connected to the same system. A very significant reduction of the inflow to the sanitary sewers can be achieved by the removal of these existing direct connections. The computer simulations indicate that the correction of the improper connections will reduce the runoff volume reaching the sanitary sewers by 50 percent.

4.3 MODELLING OF INFILTRATION - INFLOW FOR A SEWERSHED IN ALTA VISTA, OTTAWA [55]

4.3.1 Assessment and Modelling of Infiltration Inflow

In September of 1986, a heavy rainfall occurred in the Ottawa area and caused numerous basement floodings in the Alta Vista district of the city. This area had been subject to numerous basement flooding events in the past. Previous studies indicated that the probable cause of basement flooding was due to surcharge of the sanitary sewers as a result of excessive infiltration/inflow (I/I).

NOVATECH Engineering, in conjunction with Paul Wisner & Associates, were retained by the City of Ottawa to develop a master plan for basement flood relief in the Alta Vista sewershed. As part of this study, several flow gauges were installed in mid-June 1988. Rainfall and flow data on a sanitary sewer were obtained at two locations for a severe storm on June 25, 1988. Data on the sewer system were also collected and previous information available on roof connectivity was reviewed.

This information was analyzed in order to develop infiltration/inflow (I/I) design flows and hydrographs to be used for the study of relief alternatives.

It is, in general, accepted that the analysis of I/I in sanitary sewers requires measurement data:

a) Measurements at two locations, interpreted for a storm with very high intensity but dry antecedent conditions, gave, for the measured June '86 storm, an average measured value of peak infiltration/inflow of 0.3 l/s/house.

b) A hydrograph model, based on OTTHYMO (Appendix 3), was calibrated for this event and simulations were conducted at different locations along the system for the critical flooding event of September 1986.

c) Two modelling procedures (hydrographs and peak flow/house) were used for a preliminary analysis of some solutions. It was found that roof disconnection and roof leader extension may have a beneficial but limited effect on peak I/I during the design storm.

d) Weeping-tile flows in the range of 0.3 l/s/house were measured for two major storm events.

e) Basement flooding due to the September 12, 1986 storm was a result of high weeping-tile flows, superimposed on high baseflow caused by moist antecedent conditions.

f) Two peaks were measured due to the June 25, 1988 storm. The first peak occurred immediately after the peak rainfall intensity and could be attributed to the effects of runoff from directly connected roofs and driveways. The second peak occurred approximately 30 minutes to one hour after the first peak and could be attributed to the effects of weeping-tile flow and runoff from roofs not directly connected to the sanitary sewer. Both peak flows were equally intense.

g) The first peak can be eliminated by disconnecting downspouts and driveways.

h) Elimination of weeping-tile flow from the sanitary sewer would alleviate basement flooding due to sanitary sewer surcharge.

4.3.2 Alternative Remedial Measures

The analysis and development of solutions was based on a conservative design estimate of 0.5 l/s/house weeping-tile flow which was developed from the analysis of both the September 1986 and June 1988 storms. The proposed alternatives which are expected to alleviate sewer surcharging, and thus basement flooding, under those circumstances are:

a) Disconnect Downspouts

This remedial measure consists of removing downspout connections from the ground and/or providing an adequate extension to convey roof water away from the foundation. As discussed previously, the impact of this measure would be the elimination of the first, quick-response peak in the flow hydrograph. Implementation of this alternative would eliminate surcharge caused by the fast component of flow, but would not have an effect on surcharge caused by delayed weeping-tile flow. This remedial mesure was considered necessary and worthwhile and, in fact, the City of Ottawa is currently undertaking this action.

b) Disconnected Depressed Driveway Catchbasins

Most of the depressed driveways in the study area are connected to the foundation weeping tile and thus to the sanitary sewer. Disconnection of driveway catchbasins requires a gravity storm outlet and would necessarily have to be done with the disconnection of weeping tiles from the sanitary sewer. As is the case with the downspouts, disconnection of driveway catchbasins alone will not eliminate basement flooding due to delayed weeping-tile flow. Nonetheless, this alternative was considered a necessary remedial measure.

c) Sanitary Pumping Station

Under normal conditions, the system would function by gravity; however, once the flows reache the surcharge level, the pump station would pump the excessive flow via forcemain to the Rideau River Collector. In order to effectively alleviate basement flooding, this measure would have to be combined with upgrading of the local sewer. The major drawbacks of this alternative were that it was not considered to be a permanent solution, an ongoing maintenance item would be created, and storm-water inflow would not be reduced.

d) Sanitary Surge Tank

This alternative would consist of an off-line underground surge tank with pumping facilities. The measure would be short term, and it would reduce storm-water inflow. As well, local sewer upgrading would be necessary in order to effectively alleviate basement flooding. Land acquisition in an existing built-up area could also be a major problem.

e) Disconnect Weeping Tile

One obvious solution to the flooding problem is to disconnect weeping tiles from the sanitary sewer and, since surcharging is caused by upstream flows, provide protective plumbing for those houses experiencing flooding. The protective plumbing would be considered interim until such time as the disconnection could proceed far enough upstream as to eliminate surcharging. Since storm sewers are generally not deep enough to pick up weeping tile by gravity, sump pumps and forcemains to the street would be required. Due to the requirement for sump pumps, it was considered that this would be a very difficult solution to implement, especially with homeowners who are not currently flooding.

f) Storm Sewer Upgrading

This alternative consists of upgrading and constructing storm sewers at a depth suitable to intercept foundation weeping tiles by gravity. Replacement of the existing undersized trunk sewer on Fairbanks from Billings to the Moses Pepper Ditch would be necessary.

The problems with this solution include:

- To provide relief from basement flooding, protective plumbing would have to be installed in at least 50 percent of the houses.
- The solution was very costly and disruptive and did not lend itself well to phasing.
- Although modelling suggests that the storm sewers are only sized for a two-year storm, very little evidence was found that poor storm drainage was creating any significant problems; thus, complete storm sewer replacement and/or upgrading would seem inappropriate.

g) Upsizing Sanitary Trunk

It is possible to eliminate sanitary sewer surcharging caused by 0.5 l/s/house weeping-tile flow and similar circumstances to the September 1986 storm by simply upsizing the sewer to handle the anticipated flows.

Implementation of this alternative, however, would not reduce stormwater inflow.

h) Third Pipe Concept

This alternative consists of constructing a new, adequately-sized sanitary sewer to convey sanitary sewage and, for the interim, upstream weeping-tile flow, and leaving the existing sanitary system intact to convey weeping-tile flow only. The existing sanitary trunk would be rerouted to outlet to the Moses Pepper Ditch. New house service connections would be constructed and the necessary plumbing modifications made. Backwater valves would be installed outside of the basement on the new weeping-tile connection.

Construction of new sanitary sewers and creation of weeping-tile sewers from the existing pipes would be implemented on side streets as construction and phasing permits.

The estimated cost of providing a third-pipe sewer throughout the Fairbanks area was $3,400,000, which was far less costly than a complete upgrade of storm sewers in that area.

4.3.3 Recommendations and Phasing

Of the various alternatives presented, the following phasing of remedial measures were recommended for the Fairbanks-Braeside drainage area:

a) Disconnection of downspouts and add extensions throughout the entire Fairbanks-Kilborn-Braeside area.

b) Disconnection of depressed driveway catchbasins (as construction and phasing permits).

c) Implementation of the third-pipe concept, including weeping-tile connections, within the study area. In areas where storm sewers are lower than sanitary sewers or where grade permits, the weeping tile will be connected directly to the storm sewers.

4.4 ASSESSMENT OF SUB-TRUNK SANITARY FLOWS FOR THE PROPOSED BAYVIEW/SHEPPARD DEVELOPMENT IN THE CITY OF NORTH YORK [55]

1. INTRODUCTION

Paul Wisner & Associates, in conjunction with Fred Schaeffer & Associates, conducted an assessment of the proposed Bayview/Sheppard Development, which is to be located in the City of North York, south of Sheppard Avenue and east of Bayview Avenue. The total development area, which is to be located at the upstream end of the existing Vyner Greenbelt sanitary sub-trunk sewer, is approximately 9.35 ha and consists mostly of high-rise apartment buildings.

The objective of the study was to evaluate the increase in Dry-Weather Flow (DWF) due to the proposed development and to verify if flows will exceed the hydraulic capacity of the Vyner Greenbelt sanitary sub-trunk sewer as well as the Metro Toronto allocation limit for release to the East Don Interceptor system under both dry and wet-weather flow conditions.

Measurements were conducted with the System Q loggers and interpreted by means of the Manning relation (based on depths), as well as the point velocity method.

Although both methods seemed to give similar results, at one of the monitoring locations, it was found that, for low-flow depths, the difference was more pronounced. At this location, the Manning's results would be preferred for analysis due to unreliable velocity measurements in the latter part of the monitoring period.

At the other monitoring location, other problems developed due to supercritical flow conditions in a pipe with an elevated slope. In such a situation, the Manning's results were found to be unreliable, and thus the point velocity method was used for analysis at that location.

4.4.2 Analysis of Measurements

Measurements were conducted for a period of five weeks and include information for both dry-weather and wet-weather flow conditions. It was seen that the daily flow per capia was in the order of 370 l/d/c, which is less then the design criteria of 450 l/c/d.

Two significant storm events were recorded, namely; September 22, 1989 and October 19, 1989. The September 22 storm is characterized as a high-intensity, short-duration event preceded by a dry antecedent moisture period. The October 19 event, however, is characterized as a long-duration storm with a lower peak intensity. The long-duration, low-intensity portion of the storm preceding the more intense part of the October 19 event results in a wet antecedent moisture condition. Comparison of these measured events with design storms showed that they have a return period of less than two years.

The analysis of the measured hydrographs indicates that, for the event of September 22, the peak of I/I occurs at approximately 40 minutes after the peak of the rainfall. For the October 19 event, the peak flow occurs two to three hours after the peak of the rainfall. This confirms that, for an intense rainfall, such as September 22, there may be a fast response due to inflow through manhole covers or direct roof- leader connections, while for a rainfall of long duration and low intensity such as October 19, the response time is very slow due to weeping-tile contribution. Accordingly, infiltration volumes for the October 19 event were seen to be greater due to moist antecedent conditions.

At the downstream monitoring location 29, it was found that the maximum observed peak infiltration/inflow, in terms of area, was approximately 0.6 l/s/ha, while a value of 0.3 l/s/ha was observed at the upstream monitoring location. This shows a significant difference in areal contribution at each manhole and suggests a direct source of I/I such as weeping-tile flow or direct connection of roof leaders.

In terms of household contribution, Manhole 14 represents a residential area with 312 single-family units, and the maximum observed peak of infiltration/inflow was approximately 0.07 l/s/house, while the 800 units contributing to Manhole 29 represented a maximum observed peak of 0.1 l/s/house.

4.4.3 Modelling

The NASHYD sub-model (from the OTTHYMO 89 package- Appendix 3) was calibrated with measurements from the observed rainfall events and used to predict the rainfall-induced infiltration/inflow response for more severe storms such as the five-year Metro design storm (from the UMA Don Interceptor report) as well as the August 15, 1986 and August 26, 1986 events, which were critical flooding events in the City of North York. This procedure was previously used in other studies conducted by Paul Wisner in Ottawa, St. Therese, etc. After calibration, the model was used to simulate I/I hydrographs for the five-year (Metro) and August 1986 storms.

For dry-weather antecedent moisture conditions, the rainfall-induced infiltration was seem to be small (September 22 event) and, thus, the runoff coefficient was also small (C=0.015). This may be explained by the storage capacity in the soil. The response in this case was relatively fast (Tp - 0.6 hrs at Manhole 29).

For a wet-weather antecedent moisture condition, such as the October 19 event, a larger portion (8 percent) of the rainfall contributed to the runoff (C = 0.06 adjusted to exclude the wet-weather-induced infiltration), while the response was slower (Tp = 2.5 hrs). In both cases, the hydrographs were reproduced with a value of N = 2.2.

4.4.4 Effect of the Proposed Development

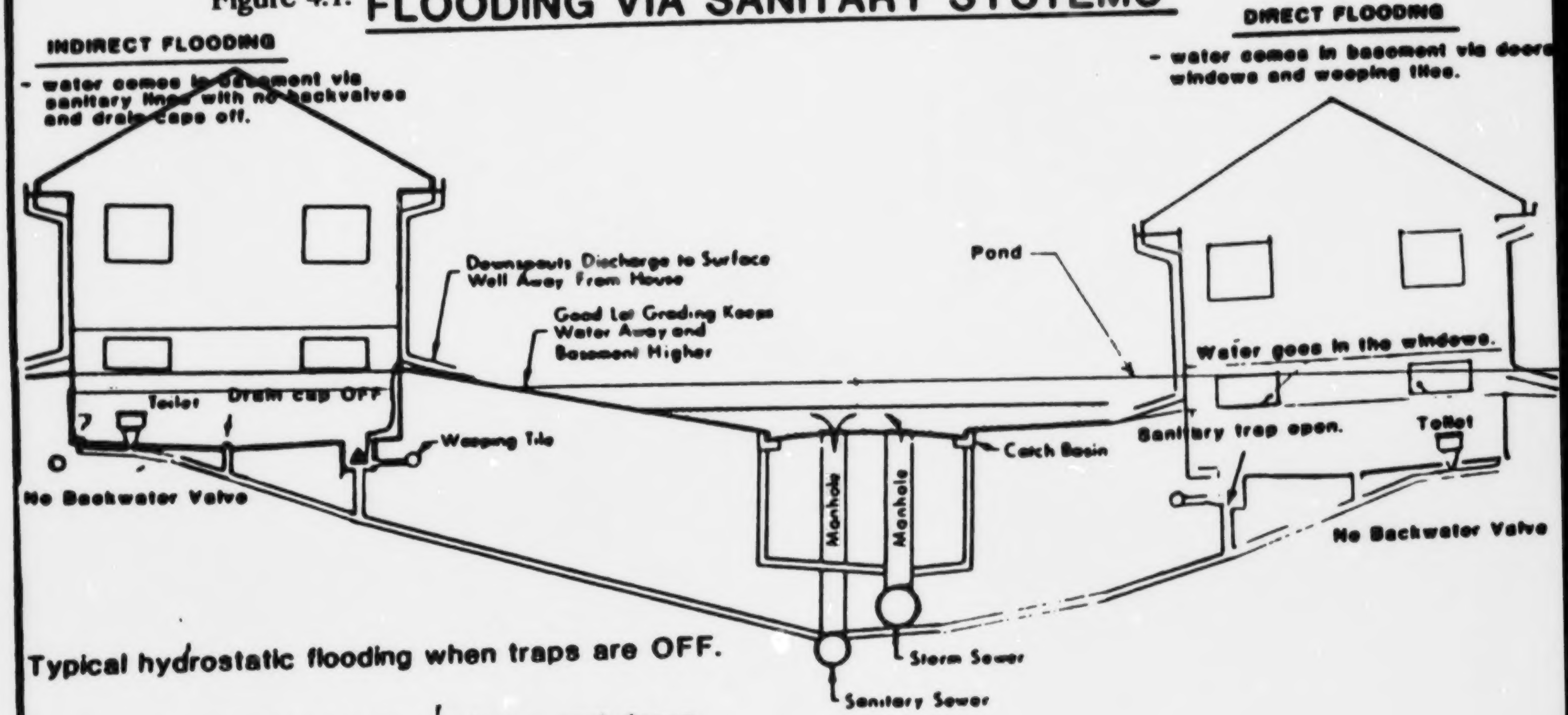
The simulated I/I hydrographs were subsequently used to determine the increase in total release to the East Don Interceptor system as well as to conduct a preliminary investigation of the Vyner sub-trunk sewer hydraulic condition. More detailed monitoring data are required to determine the distribution of I/I along the sub-trunk system. These measurements will be collected as part of Phase II of the study. It was determined from the preliminary analysis, however, that a combination of storage volumes for the control of peak flow releases to the East Don Interceptor in conjunction with increasing certain pipe capacities would be required in order to ensure that the system is safe in terms of surcharge as well as release to the Metro Toronto system.

4.5 STUDIES IN EDMONTON [10]

Many I/I basement flooding studies were conducted in Edmonton. As an example, studies in the Lago-Lindo ares showed the following (Figure 4.1):

- Street flooding at localized low spots has been identified in many locations throughout both subdivisions. Storm runoff can enter

Figure 4.1: **FLOODING VIA SANITARY SYSTEMS**


INDIRECT FLOODING
- water comes in basement via sanitary lines with no backvalves and drain caps off.
DIRECT FLOODING
- water comes in basement via doors windows and weeping tiles.
Downspouts Discharge to Surface Well Away From House
Good Lot Grading Keeps Water Away and Basement Higher
Pond
Water goes in the windows.
Toilet
Drain cap OFF
Weeping Tile
No Backwater Valve
Manhole
Manhole
Catch Basin
Sanitary trap open.
Toilet
No Backwater Valve
Storm Sewer
Sanitary Sewer

Typical hydrostatic flooding when traps are OFF.

Basement gets flooded via windows and doors.

Flood levels are transfered, via the sanitary system, to homes with no backvalves or homes with the sanitary traps left off.

the sanitary sewer through holes in the manhole lid. Such ponding can be a significant contributor to sanitary sewer inflow.

- Basement flooding during recent storms was caused by sewer line surcharging resulting from hydraulic overloading of the lines by rainfall-related infiltration and inflow (e.g., weeping-tile discharge, inflow through manhole covers).
- Extensive smoke testing of public sanitary sewer did not reveal any cross connection between sanitary and storm sewers. However, there are 11 roof downspouts connected directly into the sanitary sewer, which are a source of storm-induced inflow into the sanitary sewer.
- In some cases, lot grading does not appear to be adequately conveying rainwater away from the foundation wall area of the house. Localized ponding close, or adjacent, to the foundation wall could result, which could be a further source of infiltration water through the weeping tile.
- Weeping tile showed a very quick response to water applied at the ground surface, indicating that rain-induced infiltration from poor lot grading and poor downspout practices is a significant component of I/I.

Questionnaires were distributed and provided valuable information for the investigation. The number of questionnaires returned, however, was very small; only 18 to 20 percent of those distributed.

The field investigations can be summarized into the following comments:

- A large number of lots, 88 in Dunluce/Warwick and 232 in Lago-Lindo, have roof downspouts that discharge close to basement walls or have extensions that are ineffective in carrying rainwater from the roof away from the area immediately adjacent to the basement wall. This situation has the potential for localized ponding of water which promotes entering of water into the foundation weeping tile.
- Lot grading and relative elevations of neighbors' homes is a major concern to residents in the two subdivisions. As-built grading of lots surveyed does not correlate with that proposed at the development stage and is a potential problem with excess runoff being directed toward basement walls.
- Flooding of basements has resulted from both rainstorms with high intensity, short duration and from rainstorms with low intensity, long duration.

A short-duration storm occurred on August 20, 1988. The majority of the precipitation fell over a four-hour period. An emergency response scheme, formulated as a result of the July 5/6 storm, was in place. The plan consisted of six temporary pumping installations in each of the subdivisions' sanitary sewer systems.

In addition, several portable trailer-mounted pumps were located in each subdivision to increase relief pumping capacity.

The monitoring effort therein addressed:

- Surcharge level measurement in manholes;
- Pumping facility observation;
- Stormwater management lake observation.

A weeping-tile response hydrograph was developed for flows from the 1:5, 1:25 and 1:50-year storms in the City of Edmonton (Figure 4.2), analyses were carried out, and the collection system and pump stations in both subdivisions found to be seriously undersized under these design conditions.

The 1:50:year rainfall was proposed as the service level required. The basis for this proposal is presented as follows:

- Fifty years is a normal design service life of a collective system.
- Fifty years is the approximate period for which existing rainfall data can be used effectively to determine statistical storms.

A 1:50-year rainfall for a three-hour (short-duration) storm is proposed to determine the peak wet-weather flows, and the 24-hour (long-duration) storm was proposed to determine the storage requirements. The basis for this proposal, confirmed by sewer system analysis, is:

- Typically, a short-duration storm will produce a higher peak flow than a long-duration storm. This will result in larger pipe capacity being required for a short-duration storm than for a long-duration storm.
- Typically, a long-duration storm will produce higher volumes of flow than will a short-duration storm. This will result in larger storage facilities being required for a long-duration storm than for a short-duration storm.

Storage is required when the hydraulic capacity of an existing system is less than the peak flows generated during wet-weather flow times.

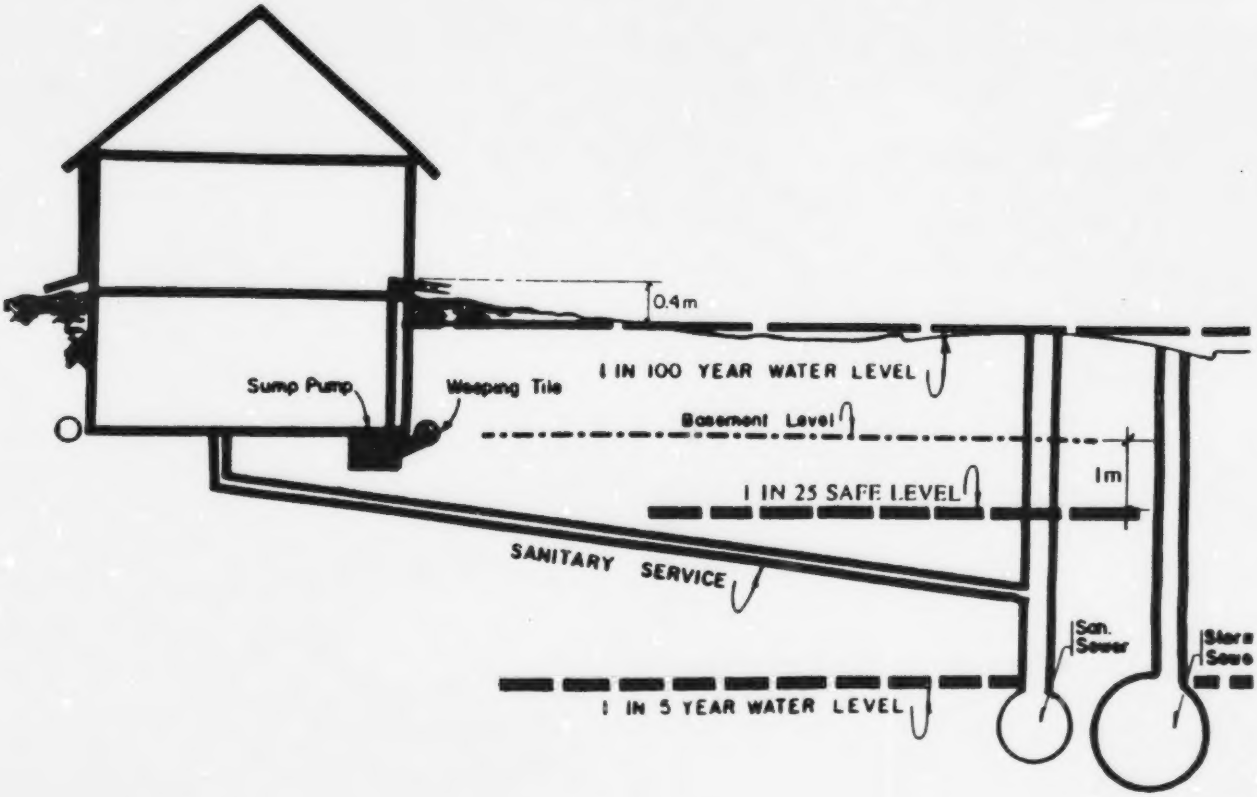

0.4m
1 IN 100 YEAR WATER LEVEL
Sump Pump
Weeping Tile
Basement Level
1m
1 IN 25 SAFE LEVEL
SANITARY SERVICE
San. Sewer
Stor Sewe
1 IN 5 YEAR WATER LEVEL

Figure 4.2: Ideal Level of Service in Edmonton MDP

An analysis was also carried out to quantify the effect on the sanitary collection systems if the foundation weeping tiles were disconnected from the sanitary sewers from all residences by installation of sump pumps.

Off-site storage by way of oversizing the downstream sanitary conveyance system was considered, rather than deep tunnel storage within the subdivision, for economical reasons.

It was also documented that the cost to rehabilitate the existing sanitary system within the subdivision boundaries for the 1:50-year service level is not significantly higher than that for the 1:25-year service level.

The study indicated that retrofitting sump pumps in existing basements will also provide a positive reduction of rain-dependent infiltration in the sanitary sewer system.

4.6 ANALYSIS OF SURCHARGE IN AN INTERCEPTOR STUDY [49]

4.6.1 General

Metro-Toronto Works Department conducted studies on the capacity of the Don and Humber interceptors which collect combined and separate sewer flows. Only the first study will be briefly presented.

In late 1987 and through 1988, 23 Montedoro-Whitney or Marsh McBirney meters were installed by staff of the Metropolitan Works Department to monitor flows in the trunk sewer system. Data for the study were available from these gauges for a period of about 10 months.

Both Montedoro-Whitney and Marsh-McBirney flow meters measure flow depth using a pressure transducer and flow velocity by electromagnetic sensors. The velocity is measured close to the sewer invert. It therefore has to be converted into an average velocity using the manufacturer's built-in "point velocity to average velocity" conversion factor based on depth of flow. Sewer flows are computed as the product of the estimated average velocity and the flow cross-sectional area given by the sewer size and flow depth (see Appendix 2).

Review of the monitoring data has shown that, in general, the depth measurements are fairly reliable, but velocity measurements are less so. It was found that good velocity measurement results were obtained for about two weeks after the monitor had been cleaned. After this, the velocity measurement component of the unit did not function reliably.

Analysis of the monitored flows commenced by determining the peak, average and minimum dry-weather flow rates. Flow measurements at all of the gauges were not always consistent; consequently, different weekly flow periods may have been used for the gauges.

4.6.2 Dry-Weather Flows

The minimum and peak dry-weather flows were compared with the average. It was found that the peak to average ratio varies between 1.04 and 1.67. The ratio of minimum-to-average flow varied between 0.42 and 0.80. In general, it was observed that the peak-to-average ratio decreased while the minimum-to-average ratio increased as the tributary sewershed area increased. This was not always consistent.

The time of peak flow increases as one proceeds downstream along the interceptor, where the Coxwell trunk discharges to the Main Treatment

Plant. The measured travel time was consistent with the travel time computed, using a measured velocity of about 1.5 m/s.

The peak-to-average-flow ratios have also been compared with the peaking factors computed with the Harmon and Babbitts formulae. It was found that the peaking ratio computed from the measured flows are less than those computed with the Harmon and Babbit formulae. The formulae give peaking ratios that range from 16 percent to 195 percent higher than the observed values.

When analyzing the existing flow measurements, it was found difficult to identify the dry-weather infiltration component. Measurements in the local systems have indicated that this component is similar to findings in other sewerage areas where infiltration rates in the range of 900 to 4 500 l/ha/d have been found. Minimum flows measured in the City of North York on the residential Grantbrook and Charlton sewersheds, when translated to infiltration rates, were found to be as low as 0 and, on average, gave values of 0.02 and 0.03 l/s/ha (1 900 and 2 600 l/ha/d), respectively.

Minimum flow conditions are presented here only for the Leslie-Steeles trunk which serves a relatively small residential area. If it is assumed that there are no wastewater contributions between 3 a.m. and 5 a.m., the flow during that interval would be representative of the dry-weather infiltration. For this trunk sewer, the low night-time flow during dry weather is 0.02 m^3/s. This translates to a unit generation rate of flow of about 0.10 l/s/ha or 8 400 l/ha/d.

Sewage generation rates were assessed using dry-weather flow measurements, land use and population data, as well as from the water usage analysis.

A sewage generation rate was computed, using the residential population upstream of each flow-monitoring location. The average rates were found to range from 354 to 1 740 l/c/d. The high rates were associated with sewersheds having high industrial components. In these areas, the water usage can be quite high and the resident population is typically lower. For this reason, the residential sewerage generation rate was only used for drainage areas which primarily service residential neighbourhoods. The sewage generation rates range from 354 to 492 l/c/d and an average value of 413 l/c/d was determined. Analysis of the dry-weather flows from the local gauges in North York indicated generation rates ranging from 209 l/c/d to 543 l/c/d in residential areas with an average of 343 l/c/d. Combining the information from both of these analyses and the water consumption rates, it was determined that a residential generation rate of 360 l/c/d (80 gpcd) was appropriate in analyzing existing conditions. This is consistent with the water consumption rates.

Dry-weather flows were also analyzed from the perspective of areal unit flow rates and found to range from 0.15 l/s/ha to 0.54 l/s/ha. This compares with a derived residential rate of 0.19 l/s/ha based on 45 persons/ha and 360 l/c/d.

Generation rates for industrial/commercial/industrial (ICI) zones were examined by reviewing the water consumption and water discharge agreements. Sewer flow measurement data have been assessed to obtain typical industrial/commercial generation rates.

In the Don sewershed, flow measurements from solely industrial areas were not available. Consequently, the industrial generation rates had to be derived. The rates were generated by deducting the daily residential flow from the total flow to obtain the industrial flow. The industrial flow, when divided by the ICI area upstream of the flow-monitoring location, yields the industrial generation rate. ICI-generation rate varies between 20 000 l/ha/d and 370 000 l/ha/d.

Industrial generation rates were found to vary considerably between sewersheds. The analysis of the water discharge agreements for specific industrial sites indicated a very wide range. Similar variations (20 000 - 37 000 l/ha/d) were found when the flow data at the monitoring locations were analyzed. Since there is such a wide variation, the industrial generation rates should be calibrated for each sewershed.

4.6.3 Rainfall-Runoff Response

Calibration of the model for WWF in sanitary areas requires knowledge of the volume of I/I entering the sewerage system. The relationship between volume of rainfall and I/I volume varies considerably for each catchment. It should be established by analysis of extensive flow records which were, in large part, not available. Consequently, a trial-and-error method using the flow records available, and the triangular hydrograph method were used to establish the rainfall - I/I relatioship and the time to peak and recession constants for infiltration and inflow. The I/I parameters selected for the sewersheds used current data (where available); engineering judgment was used to estimate the values for watersheds with inadequate or insufficient data.

To compare rainfall/runoff response, figures with rainfall hyetographs and runoff hydrographs were prepared on the same plot. The comparison was done by examining the difference between the time of peak intensity and the time of peak flow. It was found that the results were

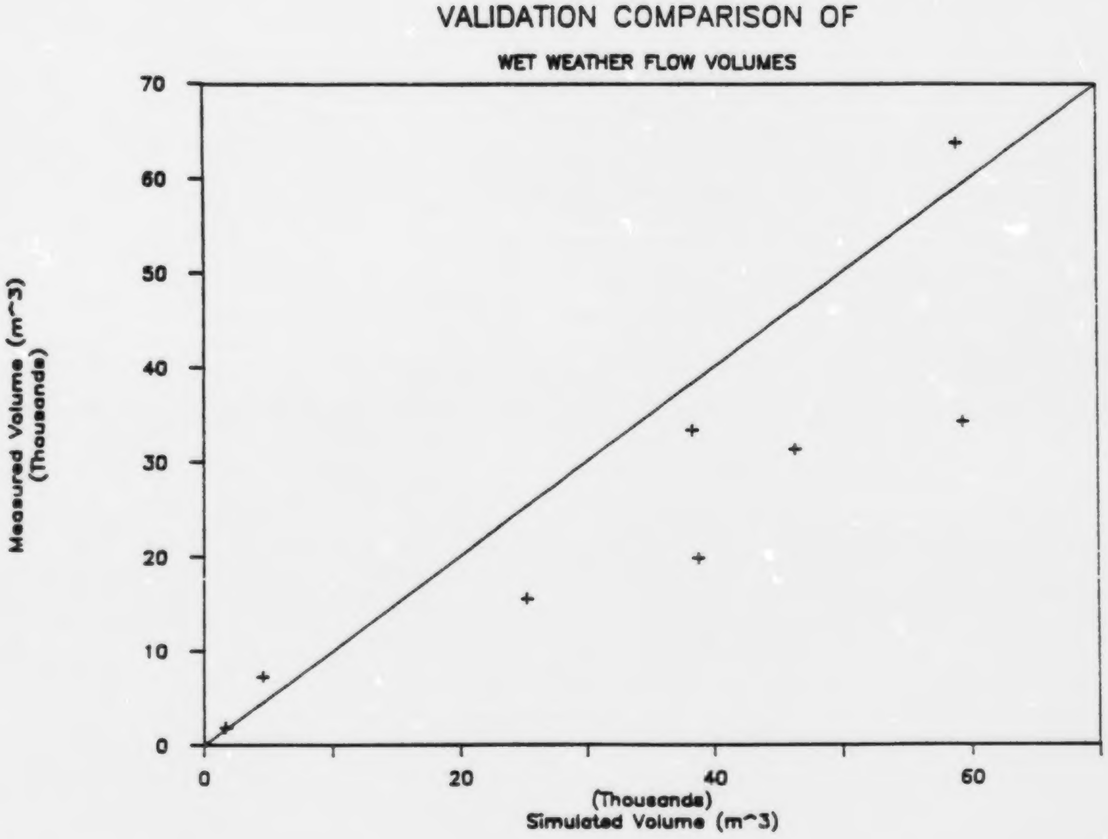


Figure 4.3: Comparison of Simulated and Measured Wet Weather Flows in the Don Interceptor Study

not always consistent; in some cases, the peak intensity occurred just at, or after, the peak flow. Reasons for this may be:

a) lack of synchronization of the flow and rain gauges since the gauges are being operated by different agencies;

b) spatial variability in the rainfall.

Further analysis of the sewer flows was done by comparing the I/I peak flows and volumes to those for dry-weather flow. The comparison shows that, even for relatively small storm events, the flow rate resulting from infiltration inflow may be more than 100 percent of the average dry-weather flow.

4.6.4 Modelling

To estimate the I/I volume, parameters 'a' and 'b' in the relation I/I VOLUME = a $(\text{Precipitation})^b$ were determined (see Appendix 1). Ratios of inflow and infiltration volumes to the total I/I volume are required, to attribute the correct volume to separate hydrographs for infiltration and inflow. The inflow was considered to be about 25.9 to 85.0 percent of the total I/I volume.

Calibration of the modified OTTHYMO model (UMASN) was performed for dry-weather and wet-weather flow conditions.

Results of the calibration runs indicate a fairly good correlation between measured and simulated flows (Figures 4.3 and 4.4).

At flow-measurement locations throughout the sewerage system, the peak dry-weather flow and the daily flow volume were compared to determine the adequacy of the calibration. The results shown in indicate a good

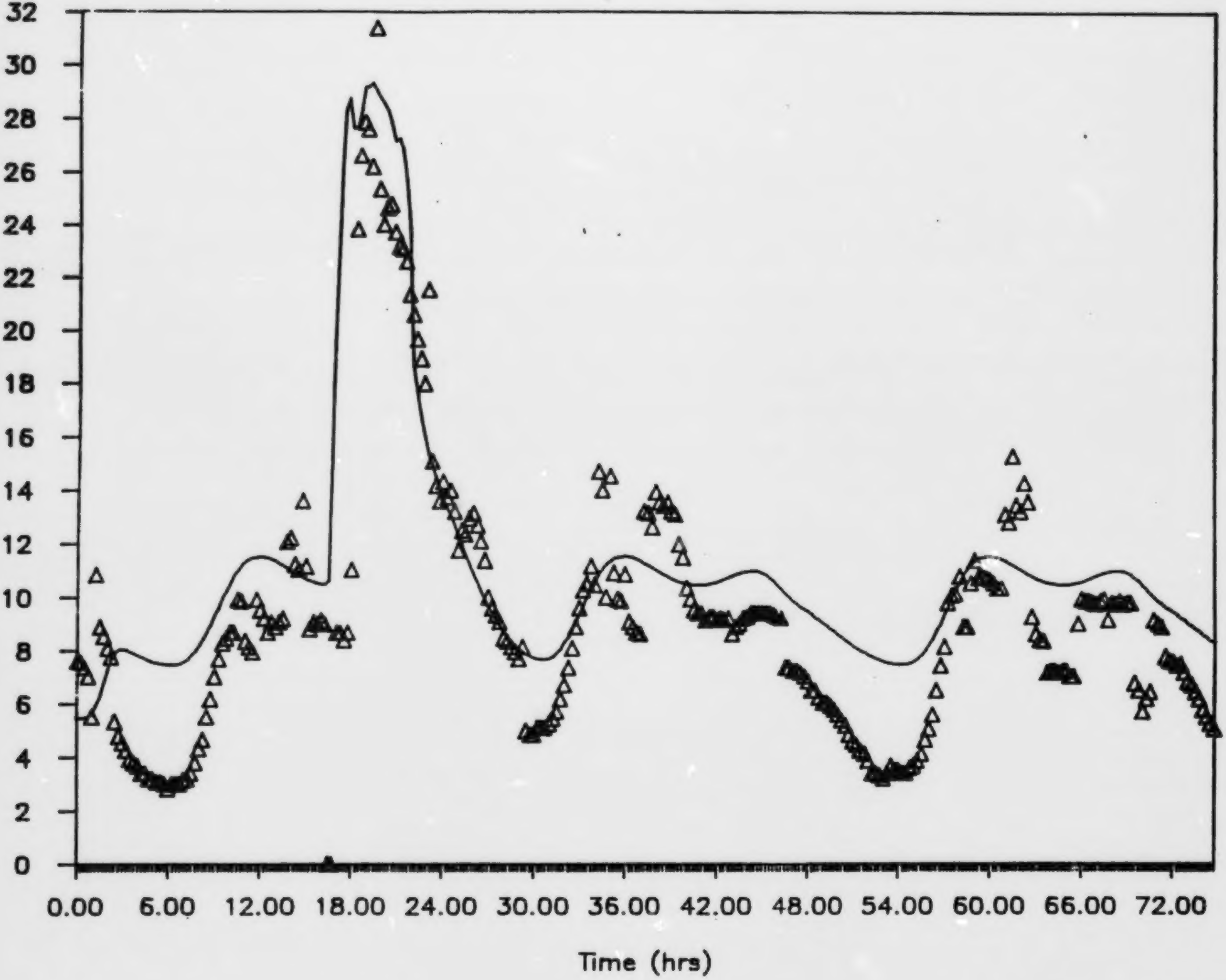


Figure 4.4: Comparison of Measured and Simulated Flows in the Don Interceptor Study (calibrated)

correlation. Both the simulated peak flow rate and total flow volume, closely approximate the measured flow rates and volumes.

The EXTRAN model analyzed flows and water depths in pipes, storages and overflows at specified time increments throughout a simulation period. To properly conduct the hydraulic analysis, it is important that the system junctions and links be correctly identified.

4.6.5 Analysis of Flooding Events

By overlapping the map showing reported basement flooding (August 15, 1986) with the sewer type map, it is evident that most of the basement flooding reports can be associated with areas where the weeping tiles are connected to the sanitary sewer. Basement flooding can also occur as a result of storm sewer backup, but this was considered beyond the scope of their study.

For the event on July 18, 1986, the simulated flows were close to capacity for most of the system. The North Toronto and Massey Creek trunk sewers exceeded capacity. For the remaining large events that occurred in August and September, 1986, the modelled flows exceeded the pipe capacity throughout most of the system.

A storm event for September 14, 1982 was selected for analyzing the operation of the whole system for various alternatives and generating flows to the sewage treatment plant. Data were available from four rain gauges in different sectors of the watershed which had rain volumes ranging from 37 mm to 55 mm. The flows computed for this event indicated that the capacity of the Massey Creek and North Toronto trunk sewers would be exceeded.

4.7 INTERFACE WITH DATA MANAGEMENT SYSTEMS [19]

4.7.1 General

Computer modelling for an area in North York was conducted for compatibility with the Data Management System, SIMS, recommended by MOE. The objective was that adequate capacity/storage could be designed to provide protection to those areas which were flooded on August 26, 1986.

A "Manhole Inspection Report" was prepared by field staff for each of the manholes and entered into the SIMS data base.

The manholes requiring some form of rehabilitation have been identified in six categories. Decisions on the work to be done were based on the field inspection of the manholes.

The street codes were obtained from the water revenue street index used for water meter reading and billing purposes. Since SIMS requires a unique code for each street, only the first number was used where more than one was listed in the water revenue street index. These codes are four-digit numbers which allows up to 9999 streets to be coded.

Digitizing from the 1:500 scale aerial plans was used in oder to efficiently and accurately develop the sanitary collection system inventory. Programs written for the Microstation CAD system were used to automatically generate manhole numbers, establish manhole northing and easting coordinates, calculate sewer lengths and generate data base files as well as graphic files. The other information available from the aerial plans, such as house numbers, street codes, inverts, diameters and pipe materials were prompted for and input during the digitizing procedure.

The digitizing programs also created the linkage between the graphics files and data base files so that the data base information can be retrieved by pointing to the graphics element (a sewer or a manhole on the screen). This allows data base reports (such as list all the manholes shown in the box) or graphics adjustments (such as turn all sewers smaller than 300 mm diameter to red) to be prepared and performed.

The graphic files have been developed using the Microstation CAD software in order to maintain compatibility with the Central Mapping Agency (CMA) Intergraph system. Separate files were created conforming to the CMA 1000 metre x 1000 metre mapping grid system to cover the entire study area.

The symbol library provided by the CMA was used to generate the manhole graphics symbols and the arrow head direction of flow indicators. These symbols have been shown on the graphics file at four times the original 1:500 scale size for plotting at 1:2500 scale as was required for this project in order to show the entire drainage area on one plan.

The text placed in the graphics files for manhole numbering, street names and street codes has also been sized for plotting at 1:2500 scale and, therefore, would be too large for plotting at the CMA base scale of 1:500.

A number of data base file structures, screen formats, inputting programs and index files were developed for this project for various purposes. The first of these are the sewer and manhole files created and linked to the appropriate graphics.

In order to check the accuracy and completeness of the data base files, a Microstation program was used to read the data from the data base files and generate a complete new graphics file showing sewers, manholes, manhole numbers and direction of flow. This was compared to the overall digitized plan, and any discrepancies were identified and corrected.

4.7.2 Data

To assess the measured flows, anticipated average and peak flows at each of the 16 monitored locations were determined based on the serviced population, various infiltration rates with a maximum of 22,500 litres per hectare per day (2,200 gal/ac/day), and the Harman peaking factor.

For the purpose of estimating the sewer flow, the following values have been considered:

Population - as per existing
Average Flow - 370 l/cap/day (80 gal/cap/day)
Infiltration - 7,850 l/ha/day (700 gal/ac/day)
- 11,200 l/ha/day (1,000 gal/ac/day)
- 22,405 l/ha/day (2,000 gal/ac/day)

Peaking Factor - Harman

These flows are not the design flows applied for the design of a sanitary sewer. The sanitary sewer design criteria for the City of North York sewers is more conservative than that considered for the study area. The design criteria for the sanitary sewers in North York is as follows:

Single-family residential - 86 persons/ha
Flow per capita - 450 l/person/day
Peaking factor - Harmon Equation
Infiltration - 25,920 l/ha/day

To identify areas of high inflow/infiltration, a flow-monitoring program was implemented for each of the 16 subcatchments. The flow monitoring at the 16 stations commenced during the first week of February, 1988 and continued to the second week of August, 1988. The flow monitoring was carried out by instrumentation to obtain continuous flow recordings within the sewer system and to monitor the effect of the spring snowmelt and rainfall within the area on the sewer system.

The majority of the rainfall data was obtained from a single rain gauge station. Reference was also made to the data obtained from two other gauges. All of these rain gauge stations are approved Atmospheric Environment Service stations. These stations record continuous rainfall on a strip chart from the tipping-bucket rain gauge.

Flow monitoring was carried out by means of modified V-notch weirs constructed of plywood and erected to fit the respective sewer manhole benching (see Appendix 2). Continuous depth recordings at every five-minute intervals of the flows over the weirs were recorded by miniature computers. They record depths on the float principle with the float cable turning a calibrated fly wheel to obtain the depths and store the values in electronic memory.

Flow monitoring at other locations was carried out using Marsh McBirney Flo-Totes. These Flo-Totes were provided and operated by the City of North York. Continuous depth recordings velocities and flows were obtained every five minutes.

4.8 PUBLIC CONSULTATION IN EDMONTON [12]

Litigations against municipalities for not providing sufficient storm protection is now becoming commonmplace. Previously, flooding arising from storms of greater than a five-year return period could be considered an act of God and the liability of a municipality was limited. The question of selection of adequate protection for relatively rare events and the implications of negligence must be debated.

Public consultation seems to be very useful for these projects. The development of services guidelines in Edmonton considered the recommendations of a public advisory group.

This Public Advisory Group (PAG) assisted the City of Edmonton in the review of Drainage Servicing Criteria.

Some of the recommendations are as follows:

- minor system sewers; capacity to transport flows equal to five-year design rainfall
- major drainage system; capacity to transport flows equal to 100-year design rainfall without producing flooding.

This is similar to the Ontario practice.

Maintenance/Repair

- Homeowner's responsibility after inspection period; including lot grading, pumps, roof leaders and downspouts, splash pads, etc.

Inspection

- City is responsible for implementing acceptable design standards at the development stage and for inspecting compliance immediately after construction and one to two years after construction.

- Permit holder is responsible for proper workmanship and completion: developer will put up a performance bond.

Enforcement

- City is responsible for having in place appropriate bylaws, licensing/permits, monitoring, sufficient inspection and enforcement systems, e.g., penalties, to ensure that standards are adequately met and maintained.

Education

- City is responsible for general awareness and understanding of the drainage system and the sharing of responsibility.
- Vendor of house is responsible for providing educational material to the home buyer to cover lot, roof and foundation features and responsibility that goes with it.
- City is responsible to develop the educational material that the developer/builder/vendor issues to the homeowner at sale.
- Ongoing education with regard to this overall program should be undertaken by the City through appropriate media and utility billing information.
- Education of City inspectors should be undertaken to ensure consistency.

Lot Grading

- Developer or permit holder is responsible to meet City standards; homeowner is responsible once the one- to two-year inspection is completed.

- Homeowner is responsible when lot grading is altered or house improvements are made requiring modified lot drainage.

Roof/Foundation Drainage

- Developer or permit holder is responsible to meet City standards; homeowner is responsible once the one- to two-year inspection is completed.

Backflow Prevention

- Compulsory installation at time of construction; responsibility of the developer or permit holder to meet City standards.

Insurance

- Homeowner is responsible for individual insurance coverage.
- City is responsible to ensure that an adequate and equitable insurance program is available, and that coverage is available for situations beyond normal insurance coverage.
- City is responsible to request that the home warranty should cover all on-lot drainage systems for an accepted period of time.

Equipment Standards

- City is responsible for establishing standards for equipment and for maintaining a list of acceptable products that meet these standards; this list is to be jointly approved by CSA, CMHC and warranty-approving agencies.
- City is responsible for ensuring that research and development is undertaken to improve the design standards and operating performance of sump pumps and other required equipment.

Legal Issues

- Resale of property should require an inspection of the drainage system, related equipment and lot improvements to ensure that all on-lot systems are functioning adequately on sale.

Information/Coordinated Support

- City is responsible for the development and distribution of information related to new standards and responsibility.
- City is responsible for ensuring that their staff is knowledgeable of policies, bylaws and standards for on-lot control.

5. ASPECTS OF EXPERIENCE IN THE U.S.A. AND EUROPE

5.1 GENERAL

In order to assess the Canadian drainage policies and practice, we review, in this section, several aspects of studies and policies in the U.S.A. and Europe.

This is not intended as a systematic state-of-the-art review but, rather, as presentation of highlights which may be used to discuss some of the present practices in this country.

5.2 REGIONAL INFILTRATION/INFLOW STUDY IN CALIFORNIA [6]

A major study on stormwater infiltration, referred to in this section as rainfall derived infiltration (RDI), was conducted in the East Bay (Oakland, California). During mid-winter, over 18 percent of the rain falling on the District service area gets into the sanitary sewer system as RDI. In some areas, more than 50 percent of the rainfall becomes RDI. The community collection systems were developed in a piecemeal fashion without proper inspection or quality control. The joints between pipes were often sealed with concrete grout which has since cracked and broken, leaving leaks in the old pipelines. Many of the original building lateral connections were not installed with fittings, but were inserted through holes broken or chipped into the main pipe. In many cases, the protruding sewer lateral stubs, break-in holes, and leaky, root-infested joints are troublesome wet-weather reminders of those antiquated building practices.

The purpose of the East Bay I/I study was to investigate the I/I problem and develop a comprehensive correction program that would eliminate overflows and bypasses at the lowest regional cost. The study took six years to complete and was performed by three independent teams made up of eight engineering consultants.

Major problems with earlier studies were caused by poor understanding of the physical nature of I/I and of the limitations of testing methods. The I/I study sought to overcome these pitfalls by concentrating on breaking down the conventional definition of infiltration and inflow into sub-components.

Monitoring and modelling were extensively applied in this study. Synthetic hydrographs were developed for each of the 300 sub-basins (small drainage basins with approximately 6 000 m of sewer) in the study area. These synthetic hydrographs were used to project wet-weather flows under design five-year storm conditions.

The major goals of a public information program was to alert residents about field work (such as smoke testing and pilot work on privately-owned laterals) and to inform the public and interested groups about the project in general. During the course of the study, over 85 presentations were made to civic groups and local governing boards. A regional workshop was conducted, and a public hearing with sessions in each community was held to fulfill Clean Water Grant requirements.

The time and money invested by the District on the public information effort returned very positive results. The public information program made a significant contribution to the successful construction of the rehabilitation project, improved customer knowledge and support of the I/I Correction Program, and greatly improved customer opinion of the East Bay Districts.

As a condition of the Clean Water Grant, a federally-funded assistance program, the communities that made up the East Bay I/I correction program had to certify that the project met its intended goal of 70-percent reduction in I/I. The steps in this typical EPA-financed I/I reduction program are illustrated in Figure 5.1.

The most significant finding of the I/I study was that inflow (for example, direct connections between storm drains and the sewer system) accounts for less than 10 percent of peak flow. Earlier studies of wet-weather problems in the East Bay had assumed that inflow was the major source of I/I and this conclusion was expected for the I/I study.

Another important finding of the I/I study was that laterals are a major source of infiltration of which RDI is the principal component. Pilot testing conducted during the study indicated that laterals may account for over 60 percent of the peak infiltration. For the pilot work, laterals were divided into upper (property line to building) and lower (sewer main to property line) proportions. Testing indicated that the upper portion was as bad, and often in a worse condition than the lower lateral.

It was determined that comprehensive rehabilitation provided the only I/I control technique that would limit the negative effect of groundwater migration to the laterals when only public sewers are rehabilitated. Based on the results of previous rehabilitation efforts (Table 5.1), one of the member Districts, the Stege Sanitary District, decided to proceed with a comprehensive rehabilitation program of not only the lower (public) lateral, but also the upper (private) lateral.

In order to perform work on the private lateral sewer as part of the rehabilitation project, the District made the decision to assume temporary ownership and control of the lateral sewer while the project was under way. By doing so, the rehabilitation contractor could be given complete

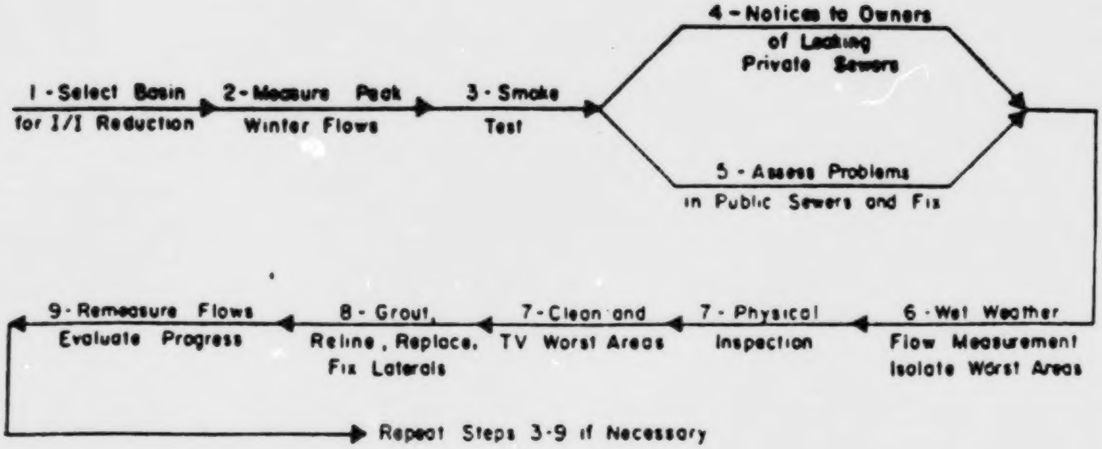

1 - Select Basin for I/I Reduction
2 - Measure Peak Winter Flows
3 - Smoke Test
4 - Notices to Owners of Leaking Private Sewers
5 - Assess Problems in Public Sewers and Fix
6 - Wet Weather Flow Measurement Isolate Worst Areas
7 - Physical Inspection
7 - Clean and TV Worst Areas
8 - Grout, Reline, Replace, Fix Laterals
9 - Remeasure Flows Evaluate Progress
Repeat Steps 3-9 if Necessary

Figure 5.1: Remedial Program for I/I Studies of Private Systems

Table 5-1: Post Rehabilitation I/I Monitoring in the USA Shows that Predicted Reduction is not Achieved

Community	GPD of I/I Predicted[1] (x 1,000)	GPD of I/I Remaining[2] (x 1,000)	Percent Reduction Predicted[3]	Percent Reduction Achieved[4]
Bell Buckle, TN	158	280	71	N/A[5]
CMSD NC	350	2,100	83	Increase
Mt. Holly, PA	491	1,010	60	23
Castle Rock, WA	185	400	82	60
Centralia, Wa	1,830	3,710	60	3
Dunsmuir, CA	71	449	99	0
Willits, CA	688	3,430	45	N/A
Shelton, WA	1,360	2,930	70	Increase
New Buffalo, MI	45	336	85	1
Amity, PA	116	742	85	24
Sussex, WI	85	899	92	7
Conyngham, PA	230	418	92	17
Mason, MI	950	1,340	52	N/A
Salem, NH	240	890	63	N/A
Vergennes, VT	124	440	79	N/A
Cortland, NY	7,000	8,370	39	Increase

[1] I/I predicted to be remaining: from Sewer System Evaluation Study (SSES) reports.

[2] Average daily I/I flow for high week, after rehabilitation: from analysis of treatment plant flow records.

[3] Percent predicted: taken from SSES report

[4] Percent reduction achieved in high week I/I flow: from analysis of treatment plant flow records.

[5] Not available.

line custody and the assumption of liability for the operations of the collection system in the rehabilitation subarea during construction. Once the construction was completed, ownership of the lateral sewer was returned to the property owner.

Post-rehabilitation monitoring results in the Stege Sanitary Districts showed an I/I volume reduction of up to 86 percent, well above the required goal of 70 percent. Prior to rehabilitation, emergency service calls relating to sewage backups and overflows had been relatively common, especially during wet-weather periods. Since the work has been completed in October 1987, no service calls had been reported in the Stege District.

5.3 STUDIES IN ILLINOIS [24, 31, 35]

Normal (population 36,000), located 130 miles southwest of Chicago, is essentially a residential community with some light, dry industry and commercial activity.

Sewer flooding became an issue in the late 1950s. Problems continued to increase as the town developed, and by the early 1970s, there were essentially four areas in which the basement flooding problems were concentrated. These areas encompassed about 1,300 residents. Of this number, some 155 experienced basement flooding at least once every three years, and some twice per year.

The older homes having basements that were not finished with drywall or paneling and carpet suffered more inconvenience than damage. As newer homes were built with finished basements, appliances, and storage closets located on lower levels, the property losses due to flooding increased in severity.

In the early 1970s, the town purchased TV sewer inspection and joint grouting equipment which was used to inspect the public mains. Sewers were cleaned and repairs of obvious problems were made. Extensive smoke testing was also conducted. Although there were a few sewer structural deficiencies, there was not a sufficient number of inflow sources found in the public system to explain the extent of the sewer backups. The engineers concluded that the major source of the problem must have been on private property.

A thorough review of the legalities governing an effort to remove private inflow sources, to find a way to relieve basement flooding temporarily, and to begin a dye testing of residences to identify actual inflow sources started in 1976.

Of the 1,310 residences tested, 741 had illegal cross-connections. It is interesting to note that, in the older problem areas, the more blatant type of cross-connection, using pipe fitting, was found (Figure 5.2). In the newer sections of town, more subtle methods of cross-connecting footing drains were used (Figure 5.3).

The council decided to pay the full cost (except for landscaping replacement) of the corrective work on private property. Corrections have averaged about $1,500 per household and the total cost to date is approximately $1 million.

When it became obvious in the 1970s that footing tile and other private inflow sources were the main source of sewer flooding, the city attorney was asked for a thorough review of the legal issues involved. Specific issues addressed by the attorney were:

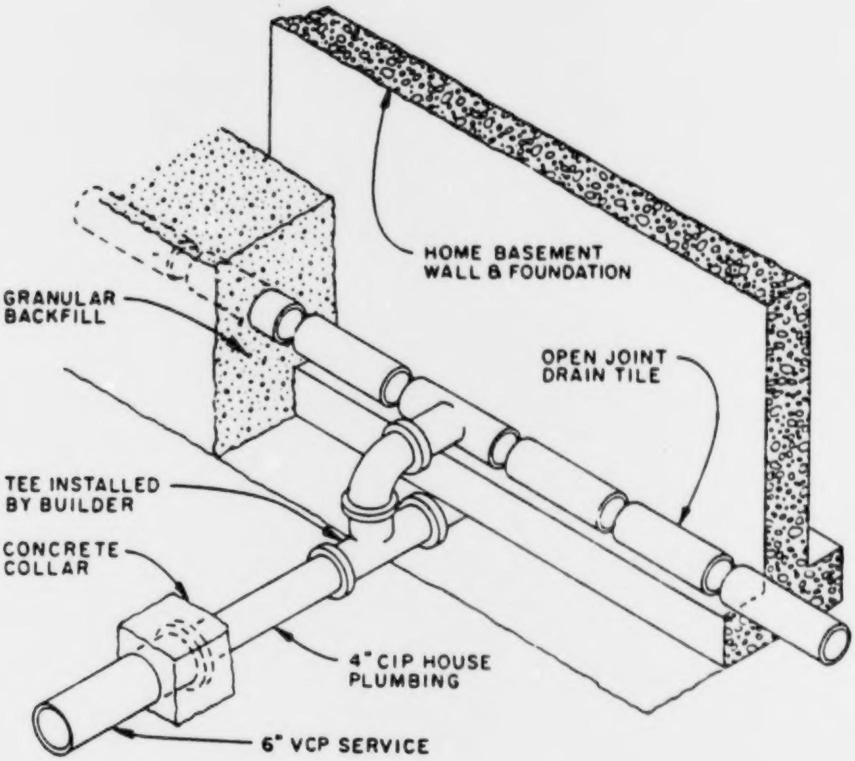

HOME BASEMENT WALL & FOUNDATION
GRANULAR BACKFILL
OPEN JOINT DRAIN TILE
TEE INSTALLED BY BUILDER
CONCRETE COLLAR
4" CIP HOUSE PLUMBING
6" VCP SERVICE

Figure 5.2: Causes of I/I in Old Private Systems

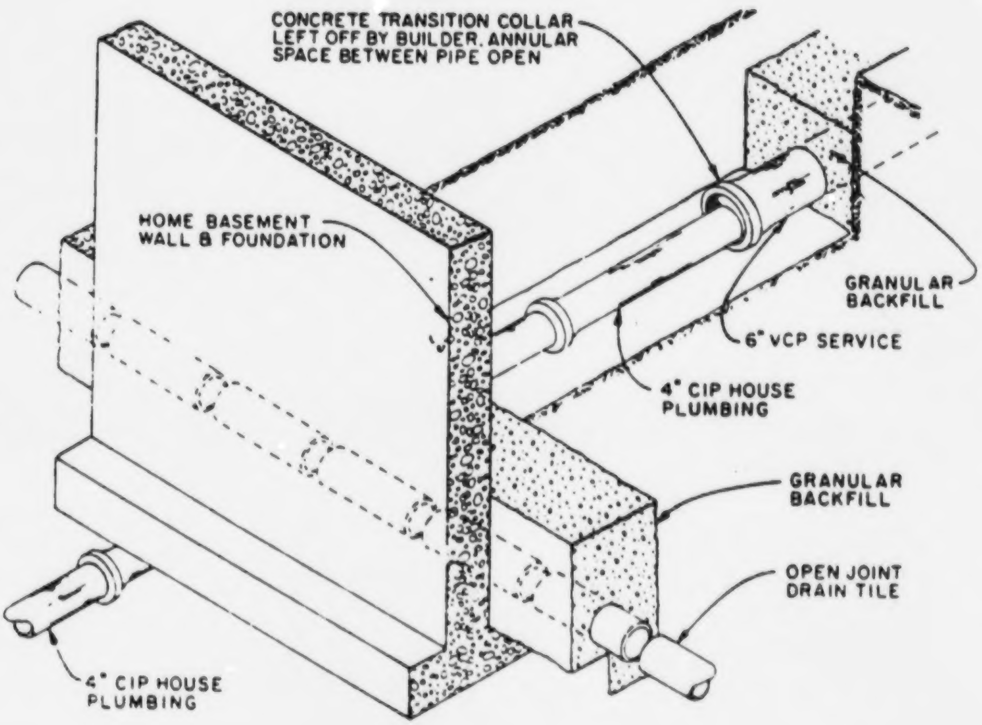

CONCRETE TRANSITION COLLAR LEFT OFF BY BUILDER. ANNULAR SPACE BETWEEN PIPE OPEN
HOME BASEMENT WALL & FOUNDATION
GRANULAR BACKFILL
6" VCP SERVICE
4" CIP HOUSE PLUMBING
GRANULAR BACKFILL
OPEN JOINT DRAIN TILE
4" CIP HOUSE PLUMBING

Figure 5.3: Causes of I/I in Recent Private Systems

1) When did it actually become illegal for a resident to connect footing drains or other inflow sources to a private sewer service line?

2) Could the town be sued for negligent testing and inspection after town ordinances prohibited the cross-connections?

3) Was the town estopped by prior actions of its inspectors?

4) Was the ordinance which the council passed to prohibit continuance of illegal cross-connections actually an "after the fact" action and, therefore, illegal?

5) Were "due process of law" standards met in the town's disconnection ordinance?

6) Did the town actually have the authority to inspect and eliminate health hazards on private property?

7) How could the town legally gain access to private properties for inspection and correction?

The council's forbidding the continued existence of an illegal connection was, in the attorney's opinion, not "after the fact." The ordinance stressed that allowing an illegal situation to continue to exist was improper as opposed to making the connections illegal retroactively. Other town codes and ordinances had already made the cross-connections illegal. This ordinance simply said that if an illegal situation existed, it must be corrected by the owner.

Legally, the town has been very successful. All warrants requested from the courts for access to private property for inspection were obtained.

The chief lesson learned in Normal's work with disconnecting private inflow sources have been in the legal and political arena. An effective public relations program must be started immediately upon undertaking an effort of this magnitude, so that people know as soon as possible the town's policy, the results of field testing, and alternatives for correcting problems. The legal review assisted the town council and staff to get to know liabilities and responsibilities for inflow source connections.

A comprehensive survey in several other small municipalities in Illinois (Elk Grove, Hoffman Estates) identified over 44,000 inflow sources in both the public sector and private sector of the sanitary sewer systems. These sources included manhole defects (13 to 45 percent of inflow), mainline sewer defects (5 to 27 percent of inflow), and private-sector sources (45 to 71 percent of inflow).

A computerized cost-effectiveness analysis of each system was conducted. The analysis considered the cost to eliminate, treat, and transport remaining inflow of incremental removal levels. The cost-effectiveness analysis resulted in development of a plan for removal of public-sector and private-sector inflow sources and construction of required relief sewers for remaining flow. Design plans and specifications were then prepared for public-sector rehabilitation and needed relief sewer construction.

Public-sector rehabilitation work included manhole repairs, cross connection removal, selected sewer line replacements, grouting of sewer lines, complete sewer line replacements, and sewer lining. Private-sector rehabilitation included disconnection of downspouts and removal of illegal storm sump pumps and combination of sump pumps from the sanitary sewer system.

Public-sector rehabilitation was funded in part by an EPA construction grant in three of the four areas. The municipalities that accepted Federal funding were required to perform post-rehabilitation flow monitoring.

Unit construction costs to remove a gallon per day (gpd) of inflow range from $0.11/gpd to $0.53/gpd.

Rehabilitation effectiveness was determined by comparing monitored peak inflow rates before and after rehabilitation. Based on monitored peak inflow and rainfall data collected prior to rehabilitation of the systems, a relationship was developed between peak inflow rate and rainfall intensity. Continuous flow monitoring was conducted for a minimum of 12 weeks in both cases during the Spring high-groundwater period. From the inflow/rainfall relationship, an inflow rate for a selected design storm event was then determined. Following rehabilitation, a similar relationship was determined, based on new flow-monitoring data, for the same storm event.

Predicted peak inflow reduction for each area was determined by considering the total flow due to inflow sources removed from the systems compared to total peak inflow before rehabilitation. Actual peak inflow reduction, as determined by post-rehabilitation flow monitoring, was very close to the predicted inflow reduction for each area.

5.4 STUDIES IN SALEM [46]

Salem Oregon's sewer system, which serves 120,000 people, is severely overloaded during winter wet-weather months. While some of the city's sewers are over 80 years old, about half of the collection system has been built since 1963. Both the older and the newer sewers are major I/I contributors. Salem has very wet winters, and nearly 25.4 in. of rainfall normally between December 1 and May 1. The leaky sewer system bypasses raw sewage, floods basements, and overtops manholes several times each winter. Altough for the last 20 years the city has conducted a major program of grouting, relining, and replacing sewers, there is little evidence that system I/I is being significantly reduced.

During the day of January 23, each manhole within the area of concentrated study was plugged on each side and the water level was allowed to rise within the manhole. If the manhole was leaking, that water was then pumped into a bucket and measured. The manhole I/I contributions were thereby determined. Although there were several manholes that leaked, none of them contributed significantly to I/I flows.

All service lateral pipes were systematically tested from the sewer main to the cleanout adjacent to the buildings. This was done by inserting a plug down the cleanout into the upper end of the service lateral pipe and by placing a mainline packer over the tee in the main. The packer was then expanded and an air test was made on the entire service lateral. Of the 31 service laterals tested, 28 failed the air test. Parallel hydrostatic water tests were run on each of the service laterals, using the packer to contain the water at the lower end of the service lateral and filling the service lateral to the top of the cleanout. In all but one case, the hydrostatic tests gave results identical to the air tests. The air tests, however, were much quicker and easier to run.

Next, repair of a number of the remaining leaking service laterals was made by using a new device, nicknamed the "snake," which had been developed precisely for the internal grouting of service lateral pipes. This device consists of a flexible rubber inflatable tube, with a device to propel it down service lateral piping from a service line cleanout. The apparatus crawls through service lateral piping, around minor bends, isolates sections of service lateral piping, air-tests the isolated sections, and grouts leaks.

It was interesting to find that flow in incompletely-repaired pipe sections sometimes increased when adjacent pipe sections were completely or nearly repaired. This "hydraulic transfer" may increase flows dramatically when the downstream segment was repaired.

Rehabilitation of just mains and manholes produced more erratic results in the two sections where only mains and manholes were made watertight, I/I flows decreased 43 percent in one section and increased 208 percent in the other.

5.5 CRITIQUE OF EPA I/I STUDIES

In the U.S., infiltration/inflow (I/I rehabilitation efforts have come under considerable criticism in the last few years. Some observers have concluded that rehabilitation has not accomplished desired results, funds have not been wisely expended, and the program should be classified as a failure.

It was stated that, after several billion dollars have been spent during the past 10 years in the United States to identify and control sewer system infiltration and inflow (I/I), most sewer rehabilitation projects under way are still not significantly reducing I/I. Generally, the sewer rehabilitation work done has been based on this theory: if we find and repair the largest and most easily-fixed I/I sources, i.e., make the most cost-effective repairs, sewer system flow will be significantly reduced. A review of past rehabilitation efforts seems to prove that this theory is not valid.

Some of the criticism will be briefly presented.

A: Extrapolation from measured flows to design values. One premise is based on the assumption that when an inflow source is located and removed from wastewater collection system, it should follow that flow at the connected treatment plant be reduced by that quantity of flow associated with the source. That premise, however, has not always been fulfilled, and a better understanding of collection system behavior can show why.

Achieving predicted levels of I/I reduction depends, in part, on techniques used to evaluate the system. For example, flow monitoring must be performed before and after rehabilitation in a manner that allows for an analytical comparison of the data.

Inflow can be correlated with rainfall intensity by using flow metering and regression analysis of multiple storm events that occur below the surcharge level. A linear relationship can be obtained to the surcharge point or until full capacity of the existing pipe is reached. The relationship also can be used to estimate a projected condition if free flow could occur beyond the surcharge level. The use of inflow rates derived from low measurements during storm events below the minimum desired level of protection invariably leads to underestimating the total potential for inflow in the collection system. This underestimate of total inflow, based on relying solely on flow monitoring data, is one of the factors contributing to the conclusion that I/I rehabilitation has been ineffective.

B: Hydraulic Interference and Surcharging. Downstream sewers often surcharge after heavy precipitation. This not only makes system analysis difficult, but can lead to faulty conclusions. When the sewer surchages, the measured I/I will not usually reflect how much that sewer can leak.

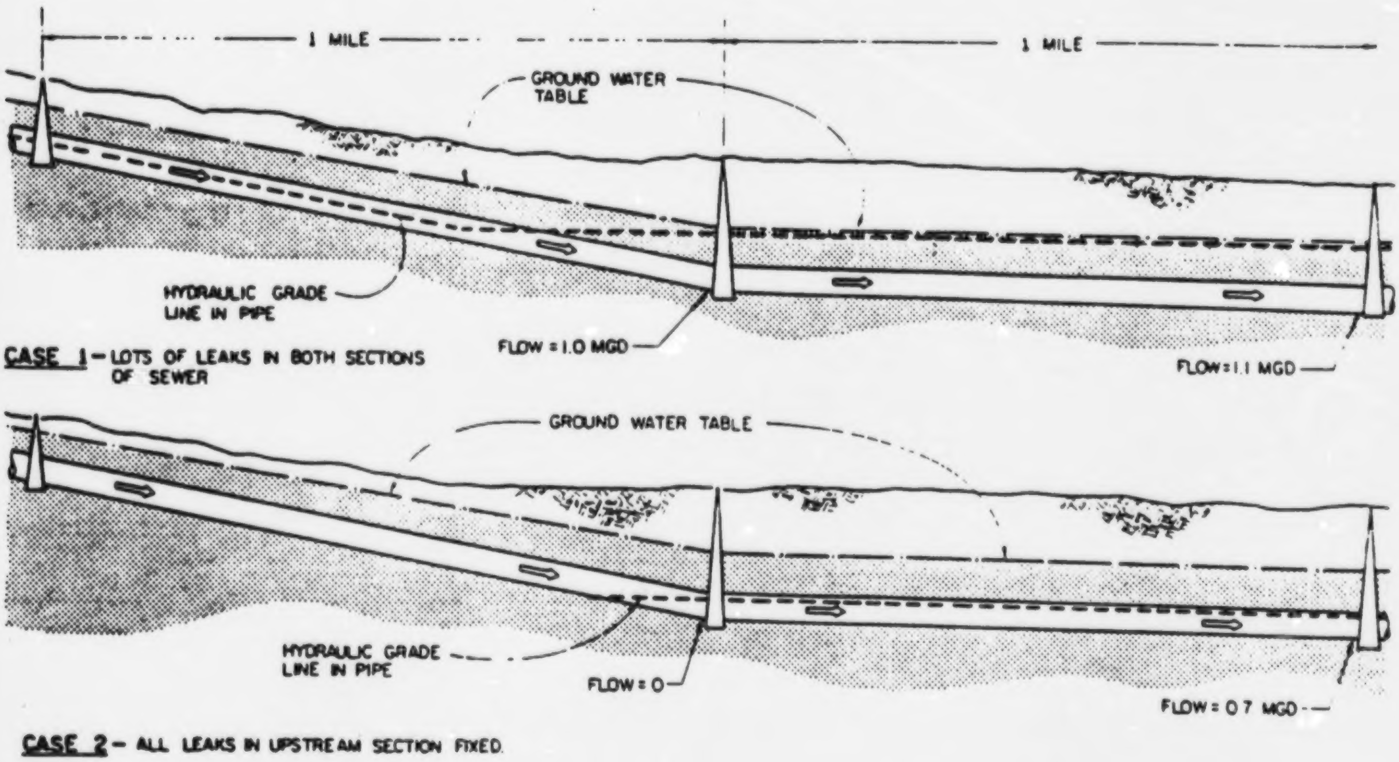

1 MILE
1 MILE
GROUND WATER TABLE
HYDRAULIC GRADE LINE IN PIPE
FLOW = 1.0 MGD
FLOW = 1.1 MGD
CASE 1 - LOTS OF LEAKS IN BOTH SECTIONS OF SEWER
GROUND WATER TABLE
HYDRAULIC GRADE LINE IN PIPE
FLOW = 0
FLOW = 0.7 MGD
CASE 2 - ALL LEAKS IN UPSTREAM SECTION FIXED.

Figure 5.4: Rehabilitation Works in the Downstream Pipe Lower the Hydraulic Grade Line and Increase I/I in the Upstream Pipe

As an example, the upstream mile in a sewer may leak badly but runs the pipe half full, and the downstream section of pipe is surcharged. Its hydraulic gradient is at about the level of the groundwater table and it is also full of leaks. Very likely, flow measurements will show that virtually all the I/I is entering the upstream one mile of pipe, while virtually no I/I enters the downstream mile. Eliminating every I/I source in the upstream mile of pipe would tend to lower the hydraulic gradient in the downstream pipe causing I/I to enter that part of the System (Figures 5.4 and 5.5). Thus EPA's normal Sewer Evaluation Survey (SSES) techniques would underestimate the corrective efforts needed to reduce I/I in this system.

C: **Vertical and Horizontal Transference.** Repairing a set of sewer leaks, particularly grouting the leaking joints in a sewer line, often results in new leaks appearing at a higher elevation. As the leaks low in elevation are stopped, the nearby watertable will tend to rise, thereby exposing other higher leaks which may not have been observed previously. Likewise, when the leaks in one section of sewer pipe are repaired, often we find that the I/I flows in adjacent or nearby sections of pipe suddenly increase.

The conclusions of the review are:

1) Renovation of a sewer system is not a "one shot" effort, but needs to be a coordinated program, often extending over several years. Surprisingly, this is often less expensive than the massive "one shot" effort.

2) It is usually more effective to spend a given sum of money completely renovating a badly leaking area than it is making scattered repairs in a large leaking system.

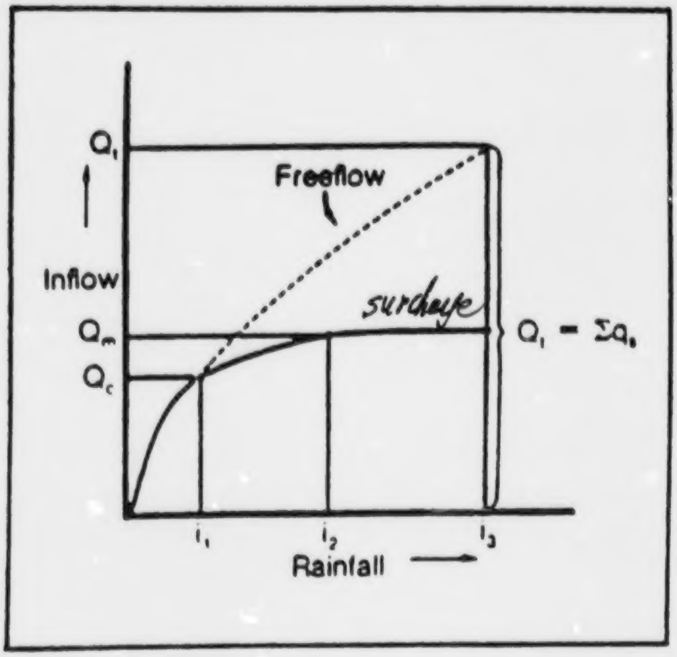


Figure 5.5: Post Rehabilitation Monitoring Shows that Remedial Measures Create Free Flow Conditions and Increase the I/I Discharge.

5.6 APPLICATION OF DECISION-MAKING MATRICES AND ECONOMIC ANALYSIS IN O'HARE STUDIES

In 1984, the U.S. Corps of Engineers undertook a study of alternative flood reduction plans for the O'Hare system of combined sewers in Greater Chicago. Current flooding problems in this system were identified, and a multi-disciplinary approach taken to evaluate mitigative measures. The overall study approached the problem from social, economic, environmental, and engineering viewpoints.

The report discussed the following:

- analysis of physical causes of high runoff, i.e., storm intensities, present and future urbanization, ground surface, hydrological aspects;
- sewer backup, i.e., configuration and conveyance of the sewer system, connections to basements, conditions of the sewer outlet (interceptors, or levels of receiving waters);
- seepage flooding - groundwater table levels, surface drainage relative to houses.

Alternative plans were formulated in an effort to develop the lowest possible cost and most efficient alternative. Initially, potential water-management measures are listed, and an initial screening process assesses their effectiveness and applicability.

This initial formulation of alternatives is performed by the study manager, environmentalists, economists, planners, hydraulic engineers, design engineers, estimators, and foundation engineers.

The measures considered for initial screening were some of the most comprehensive in the revised reports:

- rooftop detention

- disconnecting downspouts
- rainbarrel storage
- porous pavements
- temporary local ponding
- aquifer recharge
- water conservation
- vacuum or non-water-using toilets
- increased sewer capacity or on-line storage
- sewer diversion
- sewer separation
- sewer outfall improvements
- check valves
- overhead plumbing
- basement bladders
- sump pumps
- flood proofing
- elevation structures
- storage systems, such as detention basins or retention reservoirs.

A decision matrix was created, listing these variables and giving each an appropriate rating of high, medium, or low on the following criteria:

- Applicability - physical capability of implementing measure;
- Direct hydraulic impacts - the relative effectiveness of each measure;
- Engineering feasibility - current technology and engineering practices;

- Public acceptance - aesthetics, space demands, uses of water detained;
- Cost - preliminary, for comparison purposes;
- Effectiveness - ability to perform intended task;
- Reliability - maintenance, life of measure;
- Environmental impact - beneficial or detrimental impact.

For the above assessment, several of the above measures are eliminated, and the remaining pass to the final screening stage.

For the purpose of final screening, a small portion of the study area was selected to assess the effectiveness of preliminary alternatives. Modelling is performed on this test area with each measure, to assess the flood-damage reduction capacity. Effectiveness ratings and relative costs are evaluated for each measure.

In addition to costs and effectiveness, the final screening also considers environmental and social implications.

To assess the existing annual flood damage, and probable reduction in damages associated with each alternative plan, an economic assessment procedure was formulated. For the O'Hare system, economic assessment included:

1. Damages from flooding caused by sewer surcharge and seepage;
2. Economic benefits created by intensification of basement uses with less flooding.

In addition to these, overbank surface flooding and residential costs, such as transportation delays, were assessed, but these were found to be minimal in the O'Hare system.

Basic data on frequency of flooding, basement flood levels, damages reported, basement use, mitigative measures at residences, and public perceptions were gathered by comprehensive surveys completed by residents. The questionnaires employed by the O'Hare study in Chicago are included in an appendix.

The two main aspects of the procedure for the economic analysis were:

1. Identifying likely water levels that structures will be subjected to under each scenario (including existing conditions);
2. Estimating the damages.

The expected water levels in the basements of structures was analyzed by means of the survey responses of residents and hydraulic modelling of the system. Prior to the study, very little information was available on the magnitude of the flooding problem, and the various survey responses addressed the need for quantification.

Regarding water levels, it was found that the depth of flooding was a function of the flooding source (i.e., sewer backup, seepage, or overbank flow). The sources of flooding also occurred with varying frequencies. The interface between survey responses and hydraulic modelling provided the estimated water levels in basements.

Various residents had reported different local mitigative measures, such as stand pipes, check valves, sump pumps, downspouts, drain plugs, and overhead sewers. Based on the damages reported, the relative effectiveness for mitigative measures was assessed. A multiple regression analysis was applied to the survey data, producing the following relationship for flood-control devices and flood levels:

Flood Reduction Impact (inches) = 2.46 + 0.83 (Overhead sewer + 0.66 (Removed cracks) - 0.50 (Raised items) + 0.55 (Sump pump to sewer) - 0.62 (Quit usage) + 0.72 (Street check valve) + 0.29 (Stand pipe) +

0.22 (Disconnected downspout) + 0.29 (Basement check valve).

In addition, from the reported damages in the basement-use survey, a regression analysis is applied to determine the relationship between inches of water in basements and the resultant damages. In the O'Hare system report, the following relationship is reported:

$$\text{Single-family and duplex damage} = 64.12 + 141.34\,(\text{inch}) - 1.79\,(\text{inch}^2)$$

$$R^2 = 0.981$$

$$\text{Sm. Apt. Damage} = [54.71 + 161.37\,(\ln(\text{inch}^2)) + 108.8/\text{inch}]\ 4.4$$

$$R^2 = 0.952$$

$$\text{Lg. Apt. Damage} = [54.71 + 161.37\,(\ln(\text{inch}^2)) + 108.8/\text{inch}]\ 47.7$$

$$R^2 = 0.952$$

To estimate the dollar damages, the depth parameter is first estimated based upon the residence location at type of local control. The depth is then entered in the appropriate equation above.

As a final factor, a term representing the probability of sustaining damage was applied to the final result. The model estimating the depth of basement flooding did not correspond precisely to the respondents' reported damage. For example, for small predicted depths, only a small percentage of people reported flood damage. For larger floods, a larger percentage of people reported the damages. As a result, the probability of reported damage for each flood level was applied to each calculation.

The basement-use survey collected data which established the potential for intensified basement use if the public perceived adequate flood protection. If the flood problem is not regular, the household would be more likely to proceed with basement improvement, changing the value of the home. Again, the principal tools of the analysis were the survey data and multiple-regression analysis of the survey responses. The net economic benefit of this increased use was incorporated into the cost-benefit analysis of the project.

The survey conclusions were that the economic costs of restricted basement use due to flooding were $1,000 to $8,000 per house, or $278 million over the sewershed area. A flood-damage reduction program would reduce this amoung, providing economic benefits.

5.7 SOME LEGAL ISSUES IN THE U.S.A. [38]

Implementation of new solutions should consider legal issues. Legal problems in urban stormwater management may arise as a result of some action by either a land developer, builder, public agency or individual. Complaints may focus on "changes" in natural drainage flows, flood stages, topography, vegetative cover, and environmental quality, including quality of natural or man-made water bodies. Complaints may also relate to specific issues, such as: responsibility for "maintenance" of drainage facilities; liability for failure of facilities, property damages, injuries and loss of life; channel erosion and sedimentation; depreciation of the value of property, etc.

More work is required to clarify for municipal engineers the basis of liability against municipal corporations for faulty drainage.

In the U.S.A., theories of nuisance have been applied as a basis for municipal liability in appropriate instances, thereby avoiding the necessity for proof of negligence. In some cases, it has been shown that, due to a defect in the system, stormwaters were able to percolate into sanitary sewers. The natural consequence is an overtaxing of the sanitary sewer and the flooding of private homes with filth. After repeated occasions, this constitutes a nuisance in every sense of the term. It seems that the condition must have persisted for a period of time sufficient to demonstrate that it is a recurrent interference with the enjoyment of property.

A sewer system, once adequate, tends to become inadequate and insufficient as a result of continued growth of the community. In the U.S.A., it is considered that where this results in repeated instances of damage to private property, the city may be liable if, after notice of the nuisance, it fails to adopt and execute measures necessary to correct the condition. Therefore, although a city may not be liable for any initial failure to adopt an adequate drainage plan, liability will result for continued maintenance of a system which constitutes a private nuisance.

Since flooding as a result of the failure of municipal storages occurs typically after unusually heavy rains, an "Act of God" was alleged as a defense. In this respect, a city will be asked if it anticipated rainfalls such as experience shows do occur occasionally, although at infrequent intervals.

An important question is, who has the legal responsibility for maintenance of facilities for storage of stormwater runoff? Many municipalities feel that the maintenance of such structures should be public responsibility because the detention facilities are constructed for the public good. But, this is not always the case, and some local jurisdictions in the U.S.A. and Canada have required private owners to assume maintenance of stormwater detention facilities.

The legal responsibility for maintenance of detention facilities on rooftops and parking lots is even less well-documented than for facilities on ground surfaces. For the public good, it might be argued that the maintenance of such facilities should be a public responsibility. A comparison can be made with maintaining sidewalks on public rights-of-ways where the shoveling of snow is required for the public good. In most places, this is the legal responsibility of the owner or tenant of the abutting property. If the latter reasoning is applied to stormwater detention facilities on rooftops and parking lots, maintenance would become the responsibility of property owners.

One of the big questions to public officials contemplating the construction of detention facilities is the safety hazard that the facility presents, especially to children. A stormwater detention pond may be considered an "attractive nuisance." In this case, the facility owner might be held liable for any harm to persons. Because of this and the desire to protect children, many public officials require fencing the facility with the intention of relieving themselves and their agency from liability.

The U.S.A. laws governing legal liability for personal accidents attributable to safety hazards vary, depending on the jurisdictions involved and the particular circumstances surrounding each case. In Cook County, Illinois, for example, the opinion of the legal counsel of the sanitary district is that the use of fencing does not reduce the liability of the detention facility owner.

The advice and assistance of a competent attorney, qualified in urban development of SWM criteria, is considered very useful in an APWA revies. Officials of public agencies having responsibility for local drainage should review their authority for requiring specific stormwater facilities and consider the adoption of needed legislation.

5.8 ERROR ANALYSIS OF MONITORING AND APPLICATIONS IN SCANDINAVIA AND THE UNITED KINGDOM

5.8.1 CROSS LEAKAGE STUDIED IN SWEDEN [5]

An interesting study on the measurement accuracy of flow measurement equipment installed in sewage treatment plants has been carried out in Norway. Some of the findings were that the total relative error was less than +/-50% at only four of the 15 treatment plants investigated. In one treatment plant, using a v-notch weir, the relative error was as high as +186%.

The influence on accuracy due to an erroneous determination of water depth can be examined. This error is the cumulative error due to instrument installation, small variations in water depth, interpretation of a disturbed water surface by the measurement instrument and recording of the result.

It is found that there is a decrease in relative flow errors with increasing flow rates for various levels of error in level measurements.

It is therefore recommended, when estimating the obtainable accuracy in practice, to first test or estimate the error in measured water depth. Thereafter, the flow accuracy can be estimated for different water depths. Other sources of error must be added to this estimate.

Monitoring was extensively used for rainfall-induced infiltration studies in Sweden, known as "Cross-Leakage." This I/I source is common where sanitary sewers are constructed at a lower evaluation than the storm sewers. In case sewer pipes or joints are not tight, the cross-leakage flow between the two sewers will occcur as shown in the figure below.

Studies conducted in Sweden by Beckman analyzed the effect of storm water leaking out of storm sewers, which will be transported through the backfill and finally intruded into the sanitary sewers. High permeability in the trench backfill and the formation of small channels due to washout in low permeable backfill material will enhance cross-leakage. Low permeability in the material outside the sewer trench prevents exfiltrated storm water from percolating down to the groundwater table and, therefore, increases the likelihood of heavy cross-leakage flow. The study showed that this type of infiltration was measured in several sewer lines to be between 20 percent to 90 percent of the total sanitary sewer flows.

In Sweden, many investigations have studied groundwater or storm water intrusion into separate sewer systems. The need for a national compilation of the magnitude of the problem was recognized. Contributing factor of this conclusion are difficulties and accuracy problems of flow measurements in sewers.

5.8.2 SEWER MONITORING AND SIMULATION IN THE UNITED KINGDOM [4]

In the U.K., significant work in this direction was conducted by the Water Research Centre (WRC). WRC promoted the idea that it is advisable for Authorities to employ specialist flow survey Contractors to collect flow data for model verification. The recommended flow survey procedure is that of the 'saturation' survey which is to be done by covering the sewered catchment with a large number of rain gauges and flow loggers in order to record the passage of runoff hydrographs throughout the system. This approach can often be implemented only by specialist contractors as many Authorities will not have sufficient equipment. The basic Contract period for flow surveys is some six weeks, during which time it is hoped that suitable rainfall will have occurred so as to produce useful runoff hyrographs. An alternative procedure of 'sequential' flow surveys, utilizing a smaller number of flow loggers which are moved around the sewer system over a longer peridod of time, has been used successfully in Dunfermline, and is also being used in Dundee. The investigative team carrying out a sequential flow survey will gain an intimate knowledge of the way in which the sewerage system operates than will the saturation survey Contractor. Flow loggers are located on a planned basis, but, in addition, they are also moved from site to site in response to observed flow patterns which are not necessarily going to be predictable at the outset of the survey. It was also considered that well-trained in-house flow survey teams should be responsible for all aspects of data collection and also for producing the simulation model. In this way, the information gained can be most usefully converted into knowledge of the system, even though the data may take longer to collect than by a flow survey contractor. By including the benefits of having a much more complete understanding of sewer systems, sequential flow surveys can be shown to be more valuable for larger sewerage systems than saturation surveys.

In the U.K., most sewerage Authorities have responded positively to the efforts of Hydraulics Research Ltd. and the Water Research Centre encouraging the use of sewer flood simulation models to manage sewer systems. The preferred standard U.K. software is the WASSP package, at a cost several times higher than for models used in Canada.

Sophisticated computer systems have been set up to process field data and to automatically compare modelled with observed events. It was found that this type of approach requires a good understanding of the model analysis processes, in order to avoid illusory idea of precision.

MODELLING OF INFILTRATION/INFLOW

A1.1 GENERAL

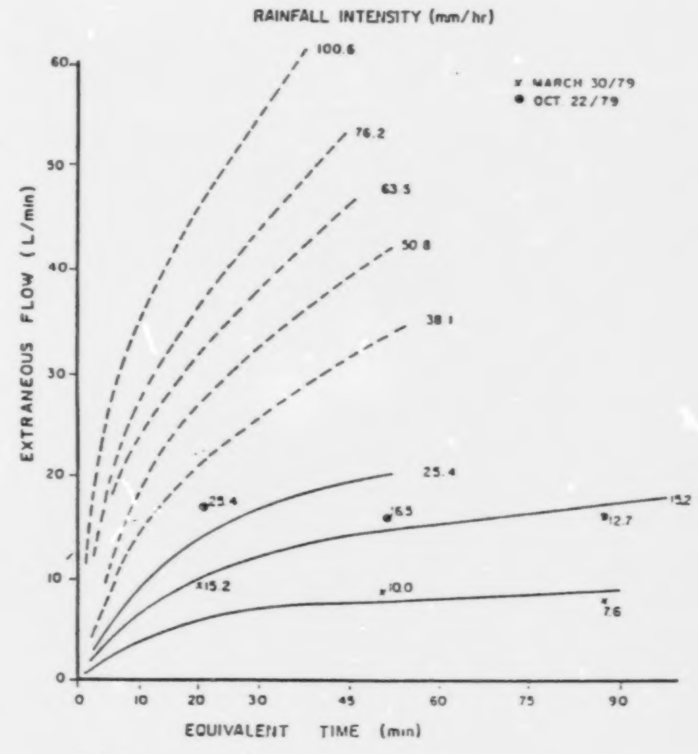
Despite the importance of the subject as shown in the preceding sections, a survey of the literature revealed that there is almost no serious publication on infiltration and inflow synthesis in sewers. This is partly because of the variability of I/I sources, the complexity of the phenomenon and the difficulties involved in describing it mathematically using fundamental physical laws. Difficulties encountered in measuring flows in sewers used to be, until very recently, a barrier toward a full analysis of extraneous flows. In fact, most of the information presented herein is based on engineering reports and personal communications which reflect merely the state of the art of current practice.

A1.2 EMPIRICAL APPROACH

More commonly than even, the rehabilitation studies of old sanitary sewers are based on steady state assumptions. Extraneous flows entering via weeping tiles, for instance, are estimated on an area basis including the normal design infiltration rate (MacLaren Inc., 1981; Paul Theil Associates Ltd., 1980; Vokes and Mullin, 1986; and others). The procedure often consisted of a trial-and-error increase of constant I/I flows with time until the measured surcharge levels of a severe storm fit the simulated values. Bearing in mind the uncertainties in; the measured surcharge levels, the rainfall data, volumes of runoff lost through flooded basements and the hydraulic model used for simulating the surcharged conditions, this approach has serious limitations.

Figure A.1.1

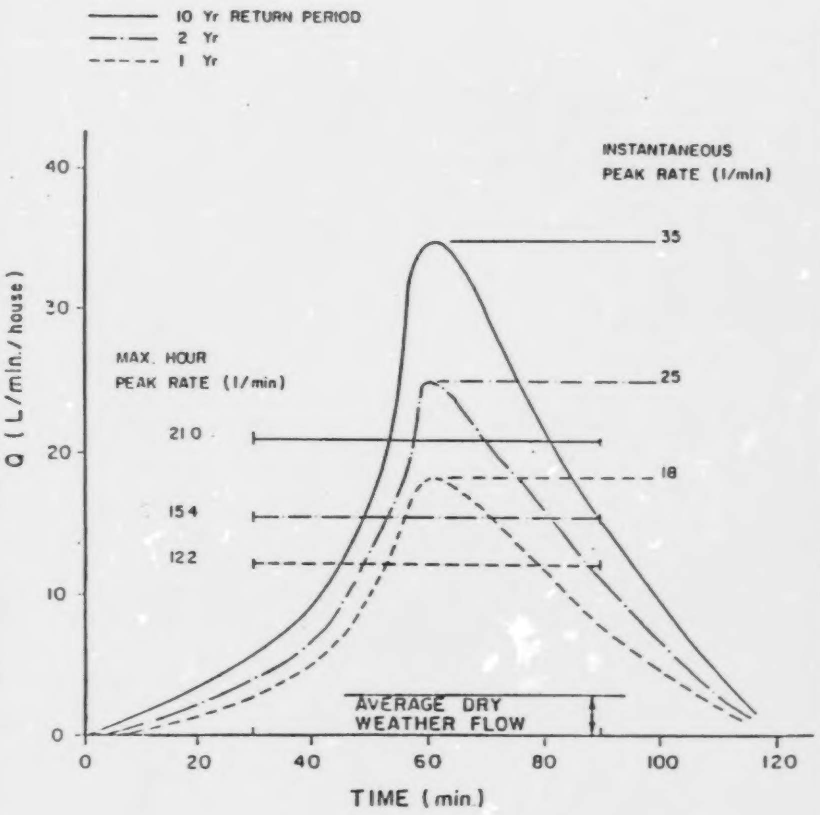
RELATIONSHIP BETWEEN EXTRANEOUS FLOWRATE, RAINFALL INTENSITY & TIME OF DURATION FOR A SINGLE HOUSE



NOTE:
1) ——— OBTAINED FROM MARCH 30/79 & OCT 22/79
2) - - - - OBTAINED BY EXTRAPOLATION

Figure A.1.2

EXTRANEOUS FLOW HYDROGRAPH



Another empirical approach (Proctor and Redfern Ltd., 1981) consists of using design curves that relate extraneous flow rate, rainfall intensity and time distribution for a typical house. These design charts are constructed based on a continuous flow monitoring program. Figure A1.1 shows a typical design chart where the time scale is defined as the total accumulated rainfall divided by the rainfall intensity at any instant. By applying these curves to each time increment of a real or design storm, an extraneous flow hydrograph can be constructed. Typical design weeping-tile responses for different return periods are shown in Figure A1.2. This methodology of modelling I/I accounts only for weeping-tile sources and has been abandoned in practice not only because it is based on a detailed house-by-house analysis (not practical for large sanitary sewersheds), but also because of its excessive data requirements.

Nogaj and Hollenbeck (1981), using data from several sewer system evaluation surveys, developed a rational technique for estimating peak inflow in a sanitary sewer system that is subject to surcharge. The technique was based on establishing a relationship between measured inflow at partial pipe flow and rainfall intensity. The best correlation was obtained based on an exponential relationship such as:

$$Q_p = aI^b$$

where a and b are coefficients, Q_p the extraneous peak flow rate, and I is an average rainfall intensity. The Nogaj and Hollenbeck model is analogous to the Rational Method and hence subject to the same type of limitations. It is actually useful in designing facilities and planning worse-case scenarious but limited in accuracy to storms of fairly short duration and high intensity. It poorly models storms that often create collection system capacity problems such as multiple day - moderate intensity rainstorms and provides only the peak point of the I/I hydrograph.

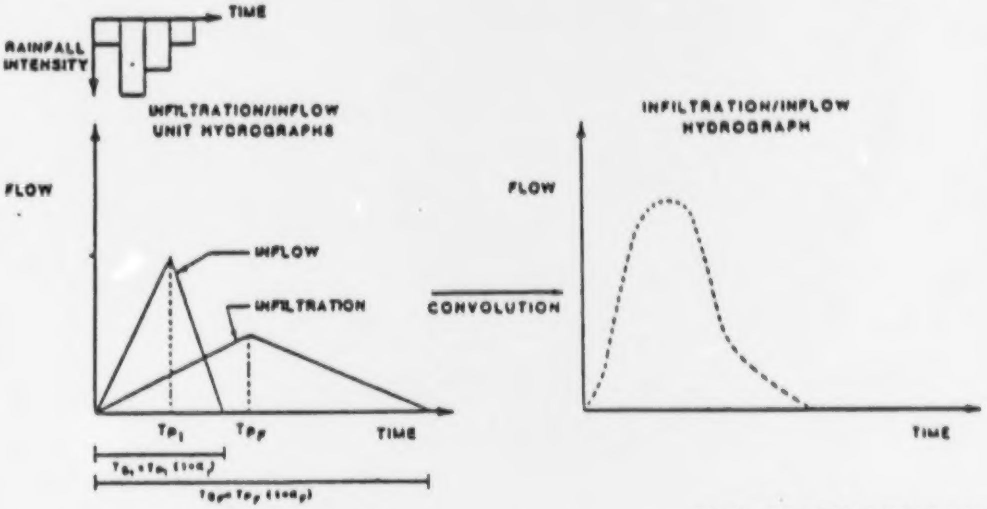
A1.3 PHYSICAL APPROACH

The many sources contributing to I/I have made the full synthesis of the system using a physical approach a somewhat tedious task. Even though, the I/I system can be separated into two distinct hydrologic subsystems, the intrusion of groundwater into pipes and direct stormwater inflows. A physical approach for modelling I/I would require synthesizing each single source individually through use of fundamental laws. Such a detailed analysis, while impractical, tedious and uneconomical, is, to an extent, possible only when one of the I/I sources is prevalent in a sewershed. A couple of attempts have been made to establish a theoretical basis of calculations of weeping-tile flows (Zarnett, 1975; Thompson, 1988; Field and Bontus, 1989).

Most of the theoretical basis for such approaches stemmed from steady-state equations relating rainfall intensity to water-table elevation and, consequently, weeping-tile flows. Such a theory was, however, originally developed for agricultural studies for removal of excess groundwater from soils with uniform conditions and widely-spaced land drains. The application of this theory to foundation drains of houses with variable soil condition is not free from serious drawbacks. Furthermore, weeping-tile flows themselves vary from site to site and even from one lot to another within the same subdivision. The main factors affecting such flows are rainfall intensity, lot grading next to basement walls, soil characteristics, roof-leader-discharge location and construction practices. Modelling systems with such conditions require some degree of conceptualization and averaging. The model also requires a large number of parameters that have to be estimated from costly field experimental procedures. On an invididual house-to-house basis, the contributing effective area, the runoff coefficient, runoff-distribution factor, the soil-bulk-percolation rate, groundwater-table elevations, the hydraulic conductivity of the backfill, its relative compaction or porosity and storage coefficients are among such parameters. This approach considers only weeping-tile flows and is deemed too detailed for large sewershed systems. Such type of models experienced only few applications by their authors, are generally not documented, and their circulation is quite restricted and hence seldom subject to wide testing and verification.

A1.4 SYSTEMATIC APPROACH

When more than one I/I source is prevalent in a subdivision, the full synthesis of extraneous flows becomes so remote that it should be discarded for practical purposes. A logical approach would be to measure observed rainfall and corresponding response flows in sewers and then attempt to establish an explicit algebraic relationship between both variables. It is then hoped that these relationships hold true within the range of conditions normally encountered in practice. Such black-box modelling procedures are widely used in the domain of hydrology. The vast complexity of some of the hydrologic systems and the inadequacy of knowledge available, and likely to exist in the future, promoted much systematic approach modelling. The main idea would be to devise a model with


TIME
RAINFALL INTENSITY
INFILTRATION/INFLOW UNIT HYDROGRAPHS
FLOW
INFLOW
INFILTRATION
T_{P_I}
T_{P_F}
TIME
$T_{B_I} = T_{P_I}(1+K_I)$
$T_{B_F} = T_{P_F}(1+K_F)$
CONVOLUTION
INFILTRATION/INFLOW HYDROGRAPH
FLOW
TIME

T_{P_I}, T_{P_F} - TIME TO PEAK

K_I, K_F - RECESSION CONSTANT

T_{B_I}, T_{B_F} - U.H. TIME BASE

INFILTRATION/INFLOW MODELLING PROCEDURE

Figure A.1.3

a minimum number of parameters determined from a fitting-curve analysis using measured data.

Modelling I/I using this theory implies lumping the various sources of extraneous flows as well as a number of houses together. The only trial on this basis, up to the author's knowledge, was first proposed by Camp-Dresser and McKee (CDM) and further developed by Fraser and Lee (1988). This methodology is based on two synthetic triangular unit hydrographs, one for each of the two components: infiltration and inflow. Rainfall data, along with unit hydrograph parameters found by calibration, can then be used to generate I/I hydrographs (Figure A1.3).

While the rationale behind the CDM technique is outstanding, their model has some limitations, namely:

- The choice of triangular unit hydrograph is arbitrary and, indeed, does not reflect the real response characteristics of the I/I components. The phenomenon is best presented by a unit hydrograph with a longer right-hand tail which portrays the long-term response of infiltration.
- The model does not consider the antecedent moisture condition nor does it account for the non-linearity of the I/I response, an aspect that can only be assessed when a sufficient and reliably monitored data bank is available.
- The effective rainfall used in the model or the portion of the total precipitation that finds its way to sewers is estimated from observed frequent rainfall and flows. This would make the extrapolation to severe storms a questionable process.

- The physical separation of extraneous flows into two different unit hydrographs implies the subjective proportioning of effective rainfall for the infiltration and inflow.

Debevoise (1988) used four years of continuously monitored data and attempted to devise a technique for modelling free surface sewer flows, using a formulation of rain-induced infiltration and a unit hydrograph for inflows. Such a model, haing a multi-event simulation nature, requires an excessive amount of data for calibration. Synthesizing the infiltration and inflow components with different formulations further complicated the situation from a practical perspective.

A1.5 CRITIQUES OF THE DESIGN STORM APPROACH

The first design storm used in engineering practices is the uniform synthetic storm derived from local IDF curves and applied in conjunction with the Rational Method to compute design peak flows. In this simplistic procedure, the critical rainfall duration is set equal to the basin's time of concentration. More complex design models allow for the variation of rainfall intensity within the storm duration. In such a case, trial-and-error procedures are required to find the critical duration and reference intensity that maximizes the peak flow for each storm sewer. The original idea of the design storm stemmed out of such an application. A design storm is defined as a rainfall pattern distributed within a critical rainfall duration for a specific return period.

The reliability of more reliable deterministic hydrologic models and long-term meteorological data as well as the advances in computers have made recent applications of the design storm a target of criticism (Adams and Howards, 1986). The concept of a synthetic storm arbitrarily assumed that the return period of the rainfall event is the same as that of the design flood and runoff volume. More generally, simulated peak flows for different return periods are assumed to have the same frequency distribution as observed peak flows generated by the same historical rainfall data from which the design storms were originally derived. Furthermore, the IDF curves, from which design storms are developed, do not consider the snowmelt component of the hydrograph. Different types of design storms have been shown to generate different shapes of runoff hydrographs. Another issue related to the subject, and often neglected, is the relation between design storms and design models. There is no design storm that can produce the statistical characteristics of real events, irrespective of the model applied. A design storm analysis is only significant when associated with a specific design model and local rainfall data. The evaluation of the validity of the various design storms available for engineering applications can be performed in two different ways.

The first being the 'direct evaluation' by means of a comparison between computed and observed peak flows. Definite conclusions using this methodology are still not available because of scarcity of experimental flow records from gauged urban areas. Paoletti (1984), Mignose et al. (1984) and Geiger (1984) followed this approach to test the performance of different types of design storms. The results showed that none of the rainfall patterns can reproduce or even approximate the real situation.

To avoid errors involved in the analysis from model approximations and difficulties arising from general lack of experimental data, an 'indirect evaluation' of design storm is still possible. This approach requires the identification of the basin behavior and a model that can be used to simulate the effects of both the historical storms and design storms. The methodology thus permits a splitting of the problem of model inaccuracies and design storm reliability in two independent parts. This type of analysis has been adopted by many authors in the literature. Amongst are Moiselo (1976), Marsalek (1978), Voorheers and Wenzel (1981, 1984), Arnell (1982), Huber et al. (1986) and Wisner and Despotovic (1988). In general, research efforts performed on the validity of different types of design storms yielded quite discordant results (Cao, 1988). The variability in the response of a particular design storm in different situations might be attributed to any or a combination of the following:

- Climatic differences affecting the main characteristics of the rainfall process;
- Model sensitivity to rainfall time distribution;
- Different basin characteristics such as area, impveriousness, slopes and network configuration;
- Sample length used for frequency analysis and its effect on the accuracy in estimating peak flows for specific return periods.

REVIEW OF MEASUREMENT TECHNIQUES

A2.1 GENERAL

As shown in the previous sections, most recent studies apply flow monitoring as a means of quantifying I/I contributions. The main techniques and errors involved are briefly discussed below:

1. Weirs and flumes are used manually or in a continuous mode.

The weir size and type determined after a field survey can be used to determine existing and anticipated flow rate. Under surcharge conditions or when free fall occurs, the weir equations and data cannot be used.

They are easy to install but have some disadvantages as follows:

- fairly high head loss
- must be periodically cleaned (unsuitable for channels carrying excessive solids)
- accuracy affected by excessive approach velocities and debris.

Flumes are essentially an open channel form of the Ventury principle. There are several types, including Parshall, Palmer-Bowlus, H-Flume and Trapezoidal configurations. Flumes are, in principle, capable of providing results accurate to within 3 to 5 percent.

Advantages of flumes are:

- self-cleaning to a certain degree
- relatively low head loss.

Gore & Storrie Limited applied, in many studies, a weir-based sewer-flow monitoring system.

The system has three main components:

1. The first is a control section with a special form of weir that ensures the relationship between level and flow rate at each station. The type of special weirs are self-cleaning and maintain sewer capacity. Their design is an integral part of the system, and it is unusual for manhole benching to be flooded at normal flows.
2. The second main component is a microcomputer that is installed in each manhole.

   Fully programmable loggers (I/I Bug) simultaneously monitor levels at numerous locations by means of digital data storage of times, levels and flows on a programmed basis. These data are then converted to flows.
3. Data are retrieved from the surface at periodic intervals, using lap-top MS-DOS-type personal computers and are placed on diskette for subsequent processing in the office. The data may also be reviewed in the field, if desired.

The last two components are similar to the Q system further described.

Raw water levels are converted to heads on the control sections and then to flows, based upon the rating-curve database.

USGS applies a pipe insert contraction (PIC) meter, developed for measuring flows in small pipes draining urban catchments. The PIC meter was successfully calibrated in the laboratory for free-surface and pipe-full conditions. The meter was also rated in situ at the field site by dye dilution and acoustic flowmeter methods.

Transient flows in large trunkline urban storm sewers may be subcritical or supercritical. A decision was made to test a modified Palmer-Bowlus flume designed to function as a venturi meter for pipe-full flow and as both a subcritical and supercritical flume during open-channel flow. An electromagnetic velocity meter is installed upstream of the flume to aid in measuring transition flows as well as flows during backwater conditions. The modified Palmer-Bowlus flume was calibrated for various flow conditions.

A gas purge-pressure transducer system was used in this study. With this system, gas (usually nitrogen) is bubbled out through an orifice and the pressure exerted by the overlying fluid is measured by the transducer. Pressure transducers, of various designs, are commercially available. The transducers used in this study varied from 10-inch to 100-inch water stage range and performed well over varied field temperature and other conditions.

A commercially available micrologger, utilizing 12-volt power supply and nine input channels, was successfully used. Apart from being a recorder, the micrologger can also be programmed to store data based on changing field conditions and to activate field measurement devices.

2) Velocity meters use the following techniques:

    a) ultrasonic methods;
    b) electromagnetic methods;
    c) mechanical current meters.

    - Using the velocity component, accurate flow data, even under surcharge conditions, can be obtained.

- Flows may reverse under surcharge conditions, some velocity meters record a positive or negative velocity.

The principles are as follows:

- Direct measurement of velocity with depth of flow can be used to calculate flow.
- Average velocity can be determined by 1-point (0.6d) or 2-point ($\frac{0.2d + 0.8d}{2}$) method.

The point-velocity method is based on the assumption that sewer discharges can be determined by empirical curves based on depth and velocity measured at a single point in a sewer-flow section for both uniform and non-uniform open-channel flow conditions, without weirs or flumes.

There are a number of sources of errors, as follows:

1) Instrument error, particularly in low depths with large full-scale probes. Normal offset errors in the probe and installation can easily be a substantial percentage of the measured full-scale depth.
2) Distorted flow profile due to pipe bends, slope changes, and other channel irregularities.
3) Disturbed flow profile due to mounting band presence (especially in small pipe "downstream" installations).

Experience indicates that theoretical prediction of sewer flow only partially contributes to overall accuracy and a practical empirical testing and correction is the best way to achieve maximum performance.

As an example, we will describe System Q, which was applied in many recent I/I studies in Canada. This system consists of:

- portable data logger with depth/velocity sensor;
- lap-top computer;
- programming/data collection software.

The measurements are done as follows:

Depth . pressure transducer
. field-calibrated.

Velocity (should be calibrated in field)
. ultrasonic sensor, highly resistant to grease/oil build-up
. self-calibrating
. immune to surface conditions.

Rainfall . tipping-bucket rain gauge
. rainfall measured in watershed being studied.

Sampler . flow-proportional sampler trigger, optional

Modem . model telemetry to retrieve data from head office, optional

Program/Interrogation is done by a lap-top computer and measured depth or velocity are applied for flow calculations automatically compensates for surcharge and reverse flow conditions).

The QBase Processing Software has the following characteristics:

- MS-DOS menu-driven;

- converts raw data to flows using Manning's or PVDM methods;
- extensive help screens available;
- can generate tables and graphs;
- make files to input Lotus 1-2-3, Symphony, dBase II and III.

DYE-DILUTION/CHEMICAL TRACERS techniques were used for calibration in several studies in Quebec. The method is simple and accurate, and it can measure flow under partially full or surcharged conditions;

The advantages are as follows:

- no entering of manholes
- provides instantaneous flow data
- independent of sewer site, dimensions, velocities and surcharging.

Some of the disadvantages are:

- samples must be analyzed ASAP
- temporary corrections required
- at least 100 sewer diameters are needed for dye-mixing before sampling.

A2.2 ERRORS IN MEASUREMENTS - AN EXAMPLE

As an example of a comparison of the use of the two methods of flow measurement, namely, the Manning's and point velocity methods, we applied two methods for the interpretation of data collected at two monitoring locations in a recent study in North York:

a) flow calculated by point velocity;

b) flow calculated with the Manning relation from measured depths.

Although both methods seemed to give similar results (Figures A2.1 and A2.2), at one of the monitoring locations, it was found seen that, for low-flow depths, the difference was more pronounced. The Manning's results would be preferred for analysis due to unreliable velocity measurements in the latter part of the monitoring period.

At the other monitoring location, other problems developed due to supercritical flow conditions in a pipe with an elevates slope. In such a situation, the Manning's results were found to be unreliable, and thus the point velocity method was used for analysis at that location.

Manning's			Point Velocity
30 Sept.	Max.	65	40
	Min.	30	13
3 Oct.	Max.	60	40
	Min.	26	8

This experience shows that the field measurements problem is a delicate one, in which one has to content not only with the limitations and accuracy of the equipment, but also with constantly varying hydraulic conditions at the monitoring site.

Each flow measurement method has its own limitations and sensitivity to deviations from ideal hydraulic conditions. Therefore, knowing the sensitivity of a specific flow measurement method in a specific sewer, combined with practical experience, is essential if reliable measurements are to be made.

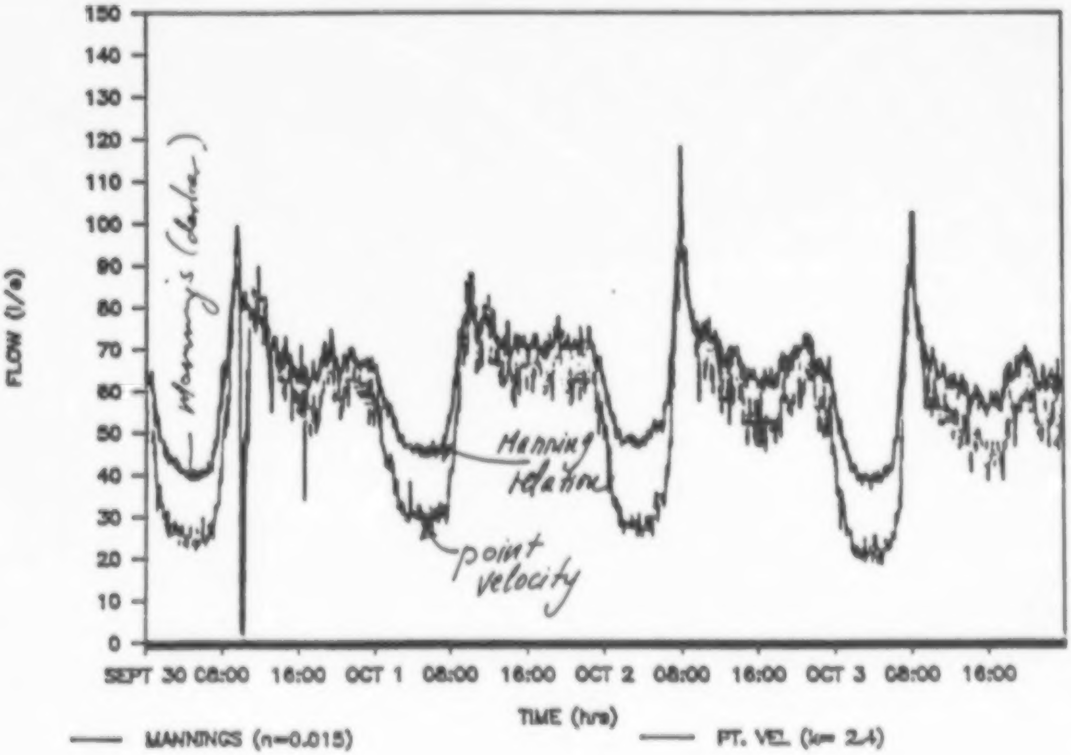


Figure A2.1: Differences in Interpretation of Flows with the same System Q Monitor

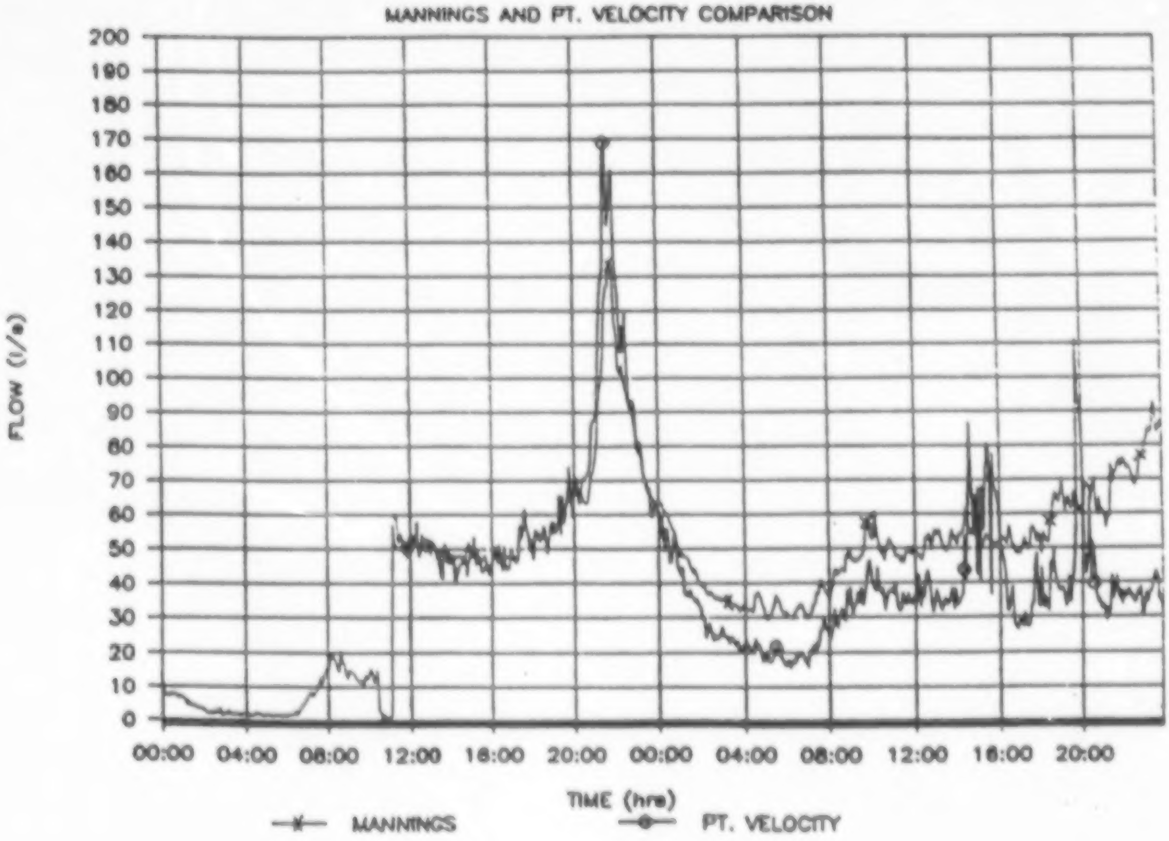


Figure A2.2: Comparison of Two Interpretations of the same Measurement with a System Q Monitor. Relative Difference is Greater for Low Flows.

APPENDIX 3

DESCRIPTION OF OTTSWMM AND OTTHYMO

A3.1 OTTSWMM

The OTTSWMM model is designed specifically for the dual-drainage concept and is based on the simultaneous analysis of the two interconnected networks formed by the storm sewers and the streets. An important feature of OTTSWMM is its capability for the hydraulic analysis of the storm inlets. By accounting for the hydraulic capacity of storm inlets, OTTSWMM can determine whether flow constricting devices are required, the locations of constrictions and the limiting capture.

OTTSWMM is composed of four main submodels:

1. a surface runoff submodel, which calculates the catchment supply rate by deducting the rainfall abstractions (depression storage and infiltration losses), and then routes the rainfall excess on the catchment surface to the drainage channels. While RUNOFF is used in the present version, users may replace it by other surface runoff submodels if desired;
2. inlet submodel, which calculates runoff captured by the storm inlets, that is, inflow into the sewers, and the by-pass flow (carry-over), which represents the major system flow. Considerations are also given to the restriction of the capture capacity of the inlets;
3. minor system submodel, which routes the inlet flows through the sewer network; and,
4. major system submodel, which routes the flows exceeding the capacity of the storm inlets through the street network.

In addition, submodels for the design/analysis of storage facilities have been provided for the minor and major system flows. Existing urban drainage models, in contrast to OTTSWMM, consider only items (1) and (3) above.

As a design tool, the mathematical and logical formulation of OTTSWMM provides, among other things, the following information:

1. sewer sizes;
2. maximum flow depth (or spread) on the streets;
3. the locations of storm inlets which may need additional constrictions; and,
4. storages for the minor and major system flows for the given outlet conditions.

Several options are provided for the evaluation of existing urban storm drainage systems, including the simulation of sewer surcharge which is carried out by means of the EXTRAN model.

A3.2 OTTHYMO

A. GENERAL

OTTHYMO 89 is a package of submodels. It uses the same technique as HYMO; namely, a series of commands generate and add, at specific points, runoff hydrographs which are then routed through channels (conduits) or reservoirs.

B. HYDROLOGIC FEATURES

Meteorological Input

The model can generate two types of design storms from intensity-duration-frequency data. It can also work as a multiple-event model by automatically conducting repeated simulations with a series of real storms. If data are available, it can also conduct simulations with an areal reduction factor, or with moving storms.

Infiltration Losses

The user can select between the traditional Curve Number (CN) method proposed by SCS, and improved CN* method developed at the University of Ottawa (in which the initial abstraction is an input parameter), the Horton relation, or a simplified proportional loss method.

Net Rainfall Runoff Transformation

The user has several options for the shape of the Instantaneous Unit Hydrographs (IUH):

- The STANDARD IUH, developed in 1978 at the University of Hannover in Germany, is similar but expands the capability of the IUH used in the URBHYD command of OTTHYMO 83, by introducing a lag of peak discharge. The peak discharge and shape vary with rainfall intensity averaged over the response time of the watershed. The Standard IUH is recommended for urban watersheds.
- The NASH IUH is a classical conceptual model given by routing through N linear reservoirs. This IUH is recommended mainly for rural watersheds.
- The NON-DIMENSIONAL SCS UH is used with a separate command in the model, and is obtained from the Nash IUH for N=5.

Base Flow

The hydrograph commands have a base flow, or dry-weather flow, option. This feature was found very useful in infiltration/inflow studies.

Reservoir Routing

The main reservoir routing routine is based on the HYMO storage-indication method.

Another command gives the approximate reservoir volume required to achieve a prescribed release rate for a given design inflow hydrograph. This simplified command facilitates outlet design and can also be used in preliminary studies where the outlet size is not required.

Diversions

The model can simulate the diversion of a portion of the flow into another watershed. The model uses the hydraulic characteristics of the diversion structure to "split" the incoming hydrograph.

Dual Drainage

The model can be used to simulate the reductions in minor system flows produced by the use of inlet control devices. A new command gives the lumped effect of several inlet control devices, and provides hydrographs for the major and minor system.

Channel Routing

SEVERAL ROUTING OPTIONS WITH DIFFERENT LEVELS OF SOPHISTICATION ARE AVAILABLE:

- SHIFT HYD is a simple translation of the hydrograph, recommended for practical applications where the peak flow attenuation is negligible, such as for short reaches, supercritical flow, etc.
- CHAN ROUTE integrates in a single command the three variable storage coefficient (VCS) routing commands of HYMO.

- PIPE ROUTE is a simplified command based on the VSC method for routing in conduits and conduit design.
- EXTRAN 89 is the most recent dynamic routing version of the SWMM model. In order to apply this sophisticated model, which can be used for surcharge simulations in conduits and routing in natural channels, INTERHYMO has an interface routine which transforms its output hydrographs into EXTRAN inputs.

REFERENCES

1. Adams, B.J. and C.D.D. Howard (1986), Design Storm Pathology, CWRJ, 11(3), pp. 49-55.

2. Alvord, Burdick and Howson Ltd. (1954), Domestic and Storm Sewers: Regina and Saskatchewan, Eng. Report prepared for the City of Regina.

3. American Public Works Assocation (1970), Combined Sewer Regulator Overflow Facilities, prepared for Federal Water Quality Administration and Department of Interior Program No. 11022 DMU, Contract 14-12-456.

4. Ashley, R.M. and C. Jefferies (1987), Sewer Simulation - Are Standard Methods Appropriate. Proceedings of the International Conference on Urban Storm Drainage, AIRH-XXII Congress, Lausanne.

5. Backman, H. (1985), Infiltration/Inflow in Separate Sewer Systems, Chalmers University of Technology, Dept. Sanitary Eng., Dissertation No. 6, Goteborg, Sweden.

6. Bishop, W.J., D.M. Diemer and M.J. Wallix (1987), Regional Infiltration/Inflow Study Solves Wet Weather Sewer Problems, Journal of Water Pollution Control Federation, Volume 59, No. 5.

7. Camp Dresser and McKee (1985), An Assessment of Storm Water Management Programs, a Final Report prepared for the Florida Department of Environmental Regulation, Tallhassee, Florida.

8. Cesareo, J.D. and R. Field (1975), Infiltration - Inflow Analysis, Jr. Env. Eng. Div., Proc. ASCE, Vol. 101, No. EE5, pp. 775-785.

9. CH2MHILL Inc. (1985), SAM: Wastewater Collection System Analysis Model User's Manual, Corvallis, Oregon 97339.

10. CH2MHILL Inc. (1988), Sewer Rehabilitation Investigations Dunluce-Warwick and Lago-Lindo Subdivisions, Final Report for the City of Edmonton Water and Sanitation.

11. City of Edmonton Environmental Services (1989), Drainage Master Plan - 1990 - 2015, A Framework For Determining Specific Project Recommendations for Edmonton's Drainage Systems Upgrading and Expansion Program.

12. City of Edmonton Environemntal Services (1989), Master Drainage Plan Option Report (Part B), Sanitary Collection Volume I, Network Evaluation, Upgrading and Expansion 1990-2015.

13. City of North York (1987), Report on Basement Flooding Due to Heavy Rains on July 18, August 15, August 26, September 10 and September 25, 1986, City of North York, Dept. Public Works, Toronto, Ontario.

14. City of Ottawa (1980), Report on the Flooding of June 16th and July 1st, 1979, Department of Physical Environmental, Engineering and Survey Branch, Ottawa, Ontario. Canada.

15. David Bromley Engineering Ltd. (1989), Northeast Edmonton Sanitary Upgrading Project, Engineering Report, Edmonton, Alberta, Canada.

16. Debevoise, N.T. (1988), The Application of the Unit Hydrograph Technique to Open Channel Flow Data, J. Water Pollution Control Fed., 60, 277.

17. Field, T. and G. Bontus (1989), Impact of Residential Weeping Tile Flows on Urban Drainage System Design, Western Canada Water and Wastewater Ass., 41st Annual Conference, Edmonton, Alberta, Oct. 4-6, 1989.

18. Fraser, H. and K.B. Lee (1988), Modelling of Infiltration/Inflow Effects on a Large Sewerage System, Proceeding of the Int. Symp. on Urban Hydrology and Municipal Engineering, Markham, Ontario, 13 - 15 June 88.

19. Gore and Storrie Ltd. (1988), Report on Needs Pilot Study, Prince Charles - Bathurst Brooke-Clye Areas, Engineering Report prepared for the City of North York, Toronto, Oct. 88.

20. Huber, W.C., B.A. Cunningham and K.A. Caender (1986), Continuous SWMM Modelling for Selection of Design Events, Int. Symp. on Comparison of Urban Drainage Models with Real Catchment Data, Dubrovnik, Yugoslavia.

21. James F. MacLaren Ltd. (1975), Review of Canadian Design Practices and Comparison of Urban Hydrologic Models, Project No. 74-8-31, project prepared for Ontario Ministry of the Environment, Pollution Control Branch, Toronto, Ontario.

22. Jankovic, M. and A. Lam (1988), Case Studies of Dual Drainage Design with ICDs in Metro Toronto, Proceeding of the Int. Symp. on Urban Hydrology and Municipal Engineering, Markham, Toronto, Ontario, 13 - 15 June 88.

23. Lam, A. (1988), Design of Small Storage Tanks for I/I Compensation, Proceeding of the International Symposium on Urban Hydrology and Municipal Engineering, Markham, Toronto, Ontario, 13 - 15 June 88.

24. Lambert, G.A. and A.J. Hollenbek (1987), New Approach Achieves Inflow Reduction in Sanitary Sewers, Public Works, USA, Sept. '87.

25. MaLaren Plansearch Inc. (1981), A Hydraulic Analysis of Sanitary Sewers in the Quetico Court Area: City of Oshawa, Report prepared for the Regional Municipality of Durham, Jan. '81.

26. MacLaren Plansearch Ltd. (1982), Final Report for the Basement Flooding Study in the City of Oshawa, Engineering report prepared for the Regional Municipality of Durhom Works Department, July '82.

27. Marsalek, J. (1978), Synthetic Sized And Historical Storms for Urban Drainage Design, Int. Conf. on Urban Storm Drainage, Southampton, England.

28. Massachusetts Department of Environmental Protection (1989), Guidelines for Performing Infiltration/Inflow Analyses and Sewer System Evaluation Survey, Division of Water Pollution Control, Boston, MA.

29. National Technical Information Service U.S. Department of Commerce (1975), Survey and Analysis of Urban Drainage Ordinances and a Recommended Model Ordinance, Georgia Institute of Technology.

30. Norgaj, R.J. (1984), I/I Rehab Success Comes with Understanding System Behaviour, Journal of Water Engineering and Management. Feb. ,84.

31. Nogaj, R.M. and A. Hollenbeck (1981), One Technique for Estimating Inflow in Sewers with Surcharge Conditions, J. Water Pollution Control Fed., 53, 591.

32. Novatech Engineering Consultants Ltd. (1984), Evaluation of Urban Drainage Methods For Basement Floodproofing, A Report prepared for Canada Mortgage and Housing Corporation under the External Research Program, March 1984.

33. Novatech Engineering Consultants (1986), Evaluation of Inlet Controls in Dual Drainage Systems, A Report prepared for Canada Mortgage and Housing Corporation under the External Research Program.

34. Paul Theil Associates Ltd. (1980), Flood Relief Study 7th Avenue Drainage Area City of Regina, Engineering Report, Regina, Saskatchewan, Canada.

35. Paulson R.I., F.S. Wylie, D.S. Anderson, F. Miles and R.T. Godrey (1984), Attacking Private Infiltration/Inflow Sources, Public Works, U.S.A., Feb. '84.

36. Pigeon J., R. Noiseaux and G. Raland (1988), Application of ICDs in Relief Sewer Studies in Laval, P.Q., Proceedings of the International Symposium on Urban Hydrology and Municipal Engineering, Markham, Toronto, Ontario, 13 - 15 June 88.

37. Pisano, C.W. (1984), An Overview of Four Inlet Control Studies for Mitigating Basement Flooding in the Cleveland and Chicago Areas, technical paper presented in Montreal, May 1984.

38. Poertner, Herbert G. (1980), Storm Water Management in the United States: A Study of Institutional Problems, Solutions and Impacts, Report prepared for the Office of Water Research and Technology.

39. Price, M. (1988), Use of Detention Ponds for the Retrofit of Storm Sewers in Scarborough, Proceedings of the International Symposium on Urban Hydrology and Municipal Engineering, Markham, Toronto, Ontario, 13 - 15 June 88.

40. Proctor and Redfern Ltd. (1981), Existing Sewer and Sewage Treatment Plant Capacity Study, Engineering Report prepared for the City of Sault Ste. Marie, Oct. '81.

41. Pupolo, J. (1989), Case Study: A Sewer Rehabilitation Needs Study, Engineering Report, M.M. Dillon Ltd., Windsor, Canada.

42. René Séguin et Associés (1988), Ville de Montreal East Plan directeur de Drainage Pluvial et Sanitaire, Rue Sherbrooke, Montreal, Quebec.

43. Schilling, Wolfgang (1986), Abfluß-Steuerung in der Stadtentwässerung, Institut für Wasser-Wirtschaft, Universität Hannover.

44. Shuttleworth, R. (1988), A Pragmatic Approach to Storm Water Management, Proceedings of the Int. Symp. on Urban Hydrology and Municipal Engineering, Markham, Toronto, Ontario, 13 - 15 June 88.

45. Stekette, C.H. (1981), Why EPS's I/I Inlet Control Program Fails, Public Works, U.S.A., July '81.

46. Stekette, C.H. and L.H. Thomas (1984), The Key to Effective I/I Control, Public Works, U.S.A., June '84.

47. Thompson, L.R. (1988), The Role of Modelling in the Determination of Inflow/Infiltration Problems in Sanitary Sewers, Proc. Int. Symp. on Urban Hydrology and Municipal Engineering, 13 - 15 June 88, Markham, Toronto, Canada.

48. Thompson, L.R. (1988), Tracking Down I/I Problems through Intensive Medium-Term Monitoring and the Role of Modelling in the Determination and Analysis of I/I Problems in Sanitary Sewers, Proceedings of the Int. Symp. on Urban Hydrology and Municipal Engineering, Markham, Toronto, Ontario, 13 - 15 June 88.

49. UMA Engineering Limited (1987), Humber Sanitary Trunk Sewer System and Treatment Plant Study, UMASAN User's Manual, Appendix 7.

50. UMA Engineering Ltd. (1989), Study of Metropolitan Toronto's Don Trunk Sewer System and Main Treatment Plant, Engineering Report prepared for the City of Metro Toronto, Sept. '89.

51. Vatagoda, S. (1988), Recent Floodings in Edmonton, Alberta, Canada and Studies for the Improvement of the Drainage System, Proceedings of the International Symposium on Urban Hydrology and Municipal Engineering, Markham, Ontario, 13 - 15 June 88.

52. Vokes, S.A. and J.F. Mullin (1986), Use of SWMM-Extran for Infiltration - Inflow Modelling of Sanitary Sewers, a Case Study of Basement Flooding in the Regional Municipality of Durham, August, Simcoe

53. Vokes, S.A. and J.E. Mullin (1988), Use of EXTRAN for I/I Modelling of Sanitary Sewers, Proceeding of the International Symposium on Urban Hudrology and Municipal Engineering, Markham, Toronto, Ontario, 13 - 15 June 88.

54. W.L. Wardrop and Associates Ltd. (1978), Storm Sewer Design Using the City of Regina Standard and The Rational Method, Engineering Report prepared for the City of Regina, Dept. of Public Works Engineering, April '78.

55. Wisner, P.E. and A. Kassem (1982), IMPSWMM Manual, Civil Engineering Department, University of Ottawa, Ottawa, Ontario, Canada.

56. Paul Wisner and Associates, Modelling of I/I for the Rehabilitation of the Alta Vista Sewer, A Report to Novatech Engineering, Nov. 1987.

57. Paul Wisner and Associates, Interpretation of Monitoring and Hydraulic Analysis for the Ottawa Interceptor Sewer, A Report to Novatech Engineering, Dec. 1989.

58. Paul Wisner and Associates, Assessment of the Vyner Greenbelt Subtrunk Sanitary Sewer Flows for the Proposed Arendsnest Development in the City of North York, A Report to Fred Schaeffer and Associates, January 1990.